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Seismic behaviour of limited ductility buildings

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Rezumat: În această lucrare se studiază cele mai importante aspecte ale comportării și proiectării seismice a clădirilor de beton armat cu ductilitate limitată, așa cum ar fi cele cu planșee ciupercă sau cu grinzi plane. Aceste tipuri de structuri sunt cele mai utilizate in Spania la proiectarea noilor clădiri și mai multe norme de proiectare din lume nu recomandă utilizarea lor în zone seismice. Comportarea seismică acestor clădiri se examinează aici utilizând o analiză neliniară incrementală (pushover analysis) care permite obținerea curbelor lor de capacitate și se compară cu comportarea clădirilor pe cadre proiectare cu ajutorul normelor spaniole și din Statele Unite ale Americii. Cele mai importante rezultate ale acestui studiu arată că doar clădirile pe cadre au suficientă ductilitate și suprarezistență pentru a garanta un bun comportament seismic. Comportamentul clădirilor cu ductilitate limitată este puternic influențat de tipologia structurală; chiar atunci când se utilizaează armatură ductilă sau o bună armare a secțiunii transversale, creștera ductilității nu este semnificativă.

Abstract: The most important aspects of the seismic design and behaviour of reinforced concrete buildings with limited ductility, like the buildings with waffled slabs or flat beams, are examined in this work. The structures with these typologies are the most used in Spain for new buildings and many seismic codes do not recommend their use in seismic areas. The expected seismic performance of these structures is studied herein by means of incremental non linear structural analysis (pushover analysis) which provides capacity curves. Their behaviour is compared with that of buildings with moment resisting frames designed according to the Spanish EHE and NCSE-02 codes and also to the ACI-318 (2005) and IBC-2003. The most important results of the study show that only the moment-resisting framed buildings exhibit sufficient ductility and overstrength to guarantee a stable seismic behaviour. The behaviour of limited ductility buildings is strongly influenced by the structural type; even if they are reinforced with ductile steel or if their confinement if improved, they exhibit slightly higher ductility.

Keywords: seismic design, structural analysis, higher ductility

1. Introduction

Among the building typologies used nowadays in the seismic areas of Spain, the most frequent have flat beams and waffled slabs (Barbat et al. 2006 and 2008). Earthquake-resistant codes, in general, and Spanish code NCSE-02, in particular, assign ductility values of two to these buildings and classifies them as restricted ductility buildings. These values are fixed by the code on the premise that buildings expressly designed to have low ISSN-1584-5990 ductility have a low capacity of energy dissipation and a non-adequate seismic behaviour. The adequacy of the response of a structure to a given seismic threat can be evaluated by using an incremental nonlinear structural analysis providing capacity curves (Erberik and Elnashai 2006), examining especially the structural ductility and the overstrength. It has to be noted that restricted ductility buildings have been not extensively studied yet using this procedure. In the past, capacity and performance-based procedures have been used mostly in evaluating the seismic behaviour of moment-

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resisting frames (Mwafi and Elnashai 2002; Fragiacomo *et al.* 2006). It has to be also mentioned that, apart from the UBC-97 and the IBC-2003, no other earthquake-resistant code directly refers to overstrength values, which are very important in the determination of response reduction factors (Vielma *et al.* 2006).

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The objective of this article is to calculate the ductility and overstrength values of buildings with restricted ductility by means of pushover analysis. The drift values corresponding to the yielding point are obtained by using the idealized bilinear form of the capacity curve (Park 1988). Once the non-linear response is determined, the benefits of improving the ductility of the steel reinforcements and the longitudinal and transversal confinement are evaluated. Finally, the non-linear response of the buildings with restricted ductility is compared with that of two moment-resisting framed buildings: one with intermediate ductility, designed according to the Spanish EHE guidelines specifications; and the other one with high ductility, designed according to ACI-318 code specifications.

2. Description of the studied buildings

To elucidate how structural typology and design influence the global response of building four buildings with structures. different characteristics have been designed and studied. The first two buildings, one of which has waffled slabs. and the other flat beams, have restricted ductility and are designed using low reduction factors. The third building, with moment-resisting frames, is designed according to the Spanish EHE guidelines and has medium to high ductility values. Finally, the fourth building with moment-resisting frames is designed according to the ACI-318 code specifications in order to fulfil requirements for high ductility.

2.1 Building with waffled slabs

The reinforced concrete building with waffled slabs (design ductility μ =2) has ribs which run along the lines that join the ends of its columns. It has three stories: the first is 4,5 m high, whereas the other two are 3,0 m high —a typical configuration for a building whose ground floor is intended for commercial use—. It has four spans parallel to the x-axis and three spans parallel to the y-axis (see figure 1a). Figure 1a also shows an equivalent frame of this building. The slab has bidirectional ribs, whereby the ribs are orthogonal to each other. The total depth of the slabs is 30 cm.

2.2 Framed building with flat beams

In the case of the structure with flat beams (design ductility μ =2), a unidirectional slab is supported on these beams (see Figure 1b). The flat beams are used both in the direction that receives the slab ribs and in that of the bracing. The story layout of the building is similar to that of the building with waffled slabs, except that the columns have been aligned with what could be defined as the resistant lines of the orthogonal frames, as observed in Figure 1b.

Just as in the case of the building with waffled slabs, the ground floor of the flat beam building is the tallest and the effect of weak ground floor is expected. However, the remaining stories have the same height and number of spans. Figure 1b shows the typical plan and elevation views of this building.

2.3 Moment-resisting framed buildings

Two buildings were designed to study the response of moment-resisting framed buildings: one according to the Spanish EHE and NCSE-02 codes (design ductility μ =4); the second one using ACI-318 (2005) and IBC-2003 (design ductility μ =6). Both frames are geometrically similar to the building with flat beams. The slabs of the building are unidirectional. Seismic design criteria are added in order to increase the column dimensions, thereby yielding a structure with strong columns and weak beams.



Figure 1. a) Typical plan and elevation view of the building with waffled slabs and b) the framed building with flat beams

3. Pushover analysis

By applying the modal analysis foreseen in the Spanish seismic code NCSE-02, the equivalent seismic forces corresponding to all the levels of the building have been calculated (Barbat *et al.* 2005 and 2007). The same inelastic spectrum was also used to calculate the seismic forces applied to the building with waffled slabs.

The aim of the non-linear analysis was to obtain a more realistic response of the buildings designed according to the linear elastic method outlined in the code NCSE-02. This allows a clear demonstration of how adequate earthquakeresistant design measures improve structural ductility, while also revealing how non-linear response challenges certain simplifications made during the elastic structural analysis.

3.1 Equivalent mechanical models of the buildings

The results were calculated using 2D models of the buildings defining for each of them representative frames. The non-linear analysis was performed with a finite element program (PLCd 1991) which enables modelling reinforced concrete as a composite material to which the Mixing Theory was applied. Discretization of the frames was performed with elements whose lengths vary in function of the column and beam zones with special confinement requirements. These confinement zones were designed according to the general dimensions of the structural elements, the diameters of the longitudinal steel, the clear of spans and the story heights.

3.2 Calculation method: pushover analysis

To evaluate the inelastic response of the four structures, a pushover analysis was performed by applying a set of lateral forces representing the seismic actions corresponding to the first vibration mode. The lateral forces were gradually increased starting from a zero value, passing through the value which induces the transition from elastic to plastic behaviour, and ultimately reaching the value which corresponds to the ultimate drift (*i.e.* the point at which the structure can no longer support any additional load and collapses). Before subjecting the structure to lateral loads simulating seismic action, it was first loaded with the gravity loads, in agreement with the combinations applied in the elastic analysis.

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The non-linear static response obtained via finite element techniques was used to generate the idealized bilinear expression shown in Figure 2, which has a secant segment from the origin to a point on the capacity curve that corresponds to a 75% of the maximum base shear (Park 1988). The second segment, which represents the branch of plastic behaviour, was obtained by finding the intersection of the aforementioned segment with another, horizontal, segment which corresponds to the maximum base shear. The use of this compensation procedure guarantees that the energies dissipated by the ideal, bilinear, system and by the more realistic finite element calculated model, are equal (see Figure 2).

In this case of a simplified non-linear analysis, there are two variables that characterize the quality of the seismic response of buildings. The first one is the structural ductility μ , defined as $\mu = \Delta_u / \Delta_y$, where Δy is the yield drift and Δu is the ultimate drift; these values can be obtained from the idealized capacity curve shown in Figure 2.

The second variable influencing on the quality of the seismic response of a building is the overstrength $R_R = V_y / V_d$, where V_d is the design base shear and V_y is the yielding base shear. The overstrength R_R is like a safety factor applied in the seismic design.

3.3 Non-linear response of the building with waffled slabs

The capacity curve of the buildings with waffled slabs, shown in Figure 2, is calculated with a mechanical model similar to the *equivalent frame* defined in the code ACI-318 (ACI Committee 318 2005). The analysis is performed by means of the finite element method, using damage and plasticity constitutive models, and the Mixing Theory (PLCd 1991; Barbat et al. 1997; Mata et al. 2007 and 2008; Faleiro et al. 2008). To control the energy dissipation and to ensure the correct behaviour of the structure, approximate mean values for strength and fracture energy were used for each constituent material (i.e. steel and concrete) (Car *et al.* 2000 and 2001).

The structural ductility of the calculated frame is $\mu = 2,91/1,85 = 1,57$, where Δy and Δu are obtained from the idealized capacity curve of Figure 2. The obtained value is lower than the design value $\mu = 2$ foreseen by the Spanish code NCSE-02 for this structural type. The overstrength is $R_R = V_y/V_d = 1,92$, that is, the structure exhibits high overstrength level. The ductility value calculated for this structural class suggests that the ductility factor values considered in the NCSE-02 earthquake-resistant code should be revised.

The low ductility response of the buildings with waffled slabs can be attributed to the formation of plastic hinges in the transition points between the abacus and the ribs of the slab at the first floor. The elements of the slabs are subjected to bending induced by gravitational loads, as well as to the demands of the seismic forces; hence, the zones which require special reinforcement are those closest to the slab-column node and to the middle of the span, where the greatest bending moments frequently appear. However, efficient confinement in the central slab zone is technically complicated. The described effect suggests a possible mechanism for structural failure during earthquakes and, consequently, the low level of ductility of the structure.



Figure 2. Capacity curve and its idealized form for waffled slabs buildings

3.4 Non-linear response of the framed building with flat beams

It is technically difficult to reinforce adequately flat beams in order to assure a ductile behaviour of the structure, what justifies the low ductility value suggested by the Spanish seismic code NCSE-02. Figure 3 shows the global response of the framed building with flat beams reaching the ultimate drift (*i.e.* the drift before total structural collapse) which, together with the yield drift, enables calculating the structural ductility. The ductility obtained for the building with flat beams is 1,54, a value which raises some concern, given that the NCSE-02 earthquake-resistant code recommends a response reduction factor of 2.



Figure 3. Idealized capacity curve for the exterior frame of the building with flat beams

The response of the building with flat beams shows that the stability of the structure depends on the behaviour of the beams. This is an important aspect to consider when deciding between using a moment-resisting frame or a frame with flat beams, given that the latter shows lower ductility values than those prescribed by the code and, consequently, can have lower response reduction factors R.

3.5 Non-linear response of the momentresisting framed buildings

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The response of the moment-resisting framed buildings was calculated and compared with the results obtained for the limited ductility structures. Figure 4 shows the capacity curve obtained for the building designed according to the Spanish codes. The curve clearly illustrates how this structural type is capable to sustain a stable ductile response, which is reflected by the high value for the final drift. Based on the idealized bilinear curve of Figure 4, a ductility factor of 5,17 is obtained —a value higher than that considered in the design, which was 4—. This means that buildings with deep beams have a ductile response to seismic forces, as well as adequate overstrength.



Figure 4. Idealized capacity curve for the moment-resisting framed building designed according to the Spanish EHE guidelines

Figure 5 show the capacity curve for the building designed according to ACI-318. The main difference between this building and the former (see Figure 4) is, by one hand, that the Spanish earthquake-resistant code limits the ductility factor for this class of buildings to four and, by the other hand, that the Spanish code requires less transversal and longitudinal reinforcement than the ACI-318 (2005) code, which enables greater dissipation capacity. The non-linear response of the studied moment-resisting framed buildings is typical for reinforced concrete low-rise structures which generally undergo plastic hinges at the base of their ground floor columns. This general tendency stems from the fact that designing buildings with strong columns and weak beams implies the predominance of gravitational loads on the beams which, ultimately, require larger cross sections than those of the columns.

4. Possibilities of improving the seisimc repsonse of buildings with restricted ductility

The results of the non-linear analysis of the buildings with restricted ductility raise the question: Can their seismic behaviour be improved at the design stage, to reach the maximum ductility values prescribed in the code NCSE-02 while maintaining the same structural type? This section discusses this possibility for buildings with either waffled slabs or flat beams, based on the pushover analysis performed using finite element models. The improved responses are finally compared with those obtained for buildings with moment-resisting frames.

With the aim of studying the influence of the steel type on the non-linear response of buildings with waffled slabs, steel with different mechanical characteristics are considered. The buildings were A.H. Barbat et al. / Ovidius University Annals Series: Civil Engineering 10, 3-12 (2008)

calculated by considering for the reinforcement either welded ductile steel (WD), whose characteristics makes it recommendable for the design of structures according to the EHE and EC-8 specifications or welded steel (W) (see Table 1). For both cases, the yield stresses B400 and B500 were considered.



Figure 5. Idealized capacity curve for the moment-resisting framed building designed according to ACI-318 (2005)

	Code			
	Eurocode 8		EHE	
Steel type	В	С	B 400 WD	B 500 WD
Yield stress fy (N/mm ²)	400 to 600	400 to 600	400	500
Ultimate stress fs (N/mm ²)	-	-	480	575
Ratio fs/fy	≥ 1,08	≥ 1,15 and ≤ 1,35	≥ 1,20 and ≤ 1,35	≥ 1,15 and ≤ 1,35
Maximum strain ɛ max (%)	≥ 5,0	≥ 7,5	≥ 9,0	≥ 8,0
Ultimate strain, ɛ u (%)	-	-	≥ 20,0	≥ 16,0

Table 1 Characteristics of the steel recommended for the design of ductile reinforced concrete buildings

The results of the pushover analyses are shown in Figure 7, which reveals that frames reinforced with ductile steel have only a slightly more ductile response than do those reinforced with non-ductile steel. Hence, the global response of the building is influenced to a much greater extent by the general configuration and the structural typology chosen than by the characteristics of the reinforcement steel. The behaviour of buildings with flat beams that are reinforced with ductile (WD) or non-ductile (W) steels, and with yield stress values of 400 or 500, has been also studied. Just as in the case of the buildings with waffled slabs, the ductile capacity of this type of building was found to be far more influenced by the structural type than by the type of steel (see Figure 7).



Figure 6. Capacity curves for the building with waffled slabs reinforced with either ductile steel (WD) or non-ductile steel (W)



Figure 7. Capacity curve for the building with flat beams reinforced with steel of different mechanical characteristics

Finally, Figure 8 shows the same results obtained for the moment-resisting frame building reinforced with different types of steel. Observe

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that, in this case, increasing the ductility of the steel leads to a major increase in structural ductility.



Figure 8. Capacity curve for the moment-resisting frame building reinforced with steel of different mechanical characteristics

5.Conclusions

A procedure of non-linear static analysis with force control has been used the yield drifts of the analyzed structures have been established using the idealized bilinear capacity curves.

Among the studied cases, only the momentresisting framed buildings exhibit sufficient ductility and overstrength to guarantee a stable behaviour, including for ductility values higher than the design ones. The obtained results also confirmed the premise that greater resistance leads to less ductility: structures modelled with B500 WD steel have higher overstrength and lower ductility than do those built with B400 WD steel.

The global behaviour of buildings with flat beams and with waffled slabs is influenced in great part by the structural type. If these buildings are reinforced with WD steel, they exhibit slightly higher ductility than if reinforced with W steel. However, for the case of moment-resisting framed buildings, the use of WD steel instead of W steel provides a substantial increase in the ductility. Moreover, the ductile response of the buildings with flat beams cannot be greatly improved via confinement of its elements; good confinement is only advantageous for buildings with momentresisting frames.

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Study on stand test up to failure of prefabricated beam elements made of prestressed and reinforced concrete

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Abstract: This paper presents in short the test results of the behaviour up to failure of a prefabricated beam for: bridge l=24 m pasaj DN1 a reinforced concrete roof beam l=11.45m interex Vaslui. The purpose of the tests was to check the quality of manufacture and to obtain necessary data for legal quality certification of these prefabricated elements.

Keywords: Breaking-point trail, quality of casting, legal registration.

INTRODUCTION

Within this study are presented the test results of up to failure of prefabricated beam elements: bridge beam L=24m, passage DN1; reinforced concrete roof beam L=11.45m interex Vaslui.

The testing purpose of the 24m bridge beam was to check the requirements of the beam at it's ultimate limits in service , respective cracks, deformation and testing the behaviour under loading to the up to failure, for legal quality certification of the production of these elements by testing the first serie and obtaining the necessarry data for the legal certification of conformity of the prefabricated elements.

The testing of the 12m roof beam had as it's purpose the checking, by tests on the stand, of the behaviour up to failure as well as the resistance of the beam for the ultimate limits in service, with special attention to the support points as well as that of the preponderant action of the shear force.

The study was carried out under the guidance of prof.dr.ing.Augustin Popaescu.

I PART

BRIDGE BEAM L=24 M PASSAGE DN1

I.1 General data

Beneficiary : KOTA KONSTRUCT SA București

General entrepreneur : LENA ENGEHARIA & CONSTRUCOES SA

Special designer : EUROPROIECT SRL ISSN-1584-5990 Roof beam tests: SC POPAESCU & CO SRL București on stand PREFA SA Brașov.

Collaborators: PREFA SA Braşov şi KOTA KONSTRUCT SA Buc.

I.2 Data regarding the tested beam

The tested beam was produced and prestressed by KOTA KONSTRUCT SA at the Băneasa building site.

Characteristics of the materials used:

Concrete resistance after 7 days was 32.87N/mm² and 52.46N/mm² after 28 days.

The checking of the concrete at the moment of testing within the Schmidt sclerometre was 50 N/mm^2 .

The wire was delivered by MECHEL Câmpia Turzii, with the quality certificate number 332/4.05.06, with resistance min. 1662 N/mm² and relaxation 5.5% at 1000 hour, 0.7 σ_{rk} , 20°C.

The prestress transfer was done after 28 days.

I.3. The test

The tests were carried out on the PREFA SA Braşov stand, consisting of a 30m beam stand and movable metal frames.

• length of prestressed beam = 24.00 m

• distance between support axes = 22.90 m

• tests carried out with 2 concentrated forces, within a distance of 3.0 m and a 2400 kN hydraulic press.

Supreme values presented by the special designer :

 M^{E} – moment of exploitation =1370 kNm, which represents M_{dec} – M_{dl} , namely moment of

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decompression ($\sigma_{binf} = 0$), exclusive moment of death load.

 M_{ld}^{E} - moment of loading at the condition limit of expoitation -1500 kNm, representing $M_{crack} - M_{dl}$, namely moment of apparition of the cracks, considering $R_t = 1.65$ N/mm², exclusive moment from death load.

The arrow under the action of the moment of exploitation was 53.7mm at the middle of the opening.

I.4. Measurements

• arrows at the middle of the opening and under the force P with the comparator with wire and there were made diagrams of the arrows.

• apparition, opening, enclose of the normal magnifier glass with 0.1-0.02mm gradation and there were made measurements of the arrows.

• specific deformation of the superior and inferior fibers of the central section and under the forces with mechanical deformetre.

• globally photos and in detail.

I.5 Results and interpretation

The testing was effectuated by the application of the forces in cycle (with unloading) by considering the stairs after the supreme values presented by the special designer.

 $M_{dec}-M_{dl}$ = 1370 kNm and $M_{crack}-M_{dl}$ = 1500 kNm.

There was calculated the capable moment at the ultimate limits of resistant:

 M_{cap} ULR = 2423 kNm, respectively $M_{ULR} - M_{dl} = 1899$ kNm, for which correspond a force P = 192 kN. There was estimated the arrow at the $M_{crack} - M_{dl}$ considerating the ideal section, resulting f = 44.7 cm.

The firstcycle of the upload was guided till P = 127 kN. At the moment of unloading the permanent arrow was 2.8%.

The second cycle of upload was guided till the crack apparition at P = 158kNm, respectively

 $\Delta M = 1594 kNm > 1500 kNm \text{ (supreme value)}$ and in the continuation till $\alpha_f = 0.1 - 0.15 mm$, loading corresponding to 0.98 M_{capULR}. At this loading the arrows was 38.5mm (1/600) < <44.7 cm.

The closure of the cracks was at P = 78 kN, ΔM = 776 kNm and M = 1300 kNm, and the reopening of the cracks prodused at P = 132 kN, ΔM =1313 kNm, M = 1838 kNm.

 $\Delta M = 1313$ kNm represented 0.96 of $M_{dec} - M_{dl} = 1370$ kNm.

The permanent arrow was 3.8% from the arrow at this cycle < 10% admissible.

The loading was continued till P = 340 kN, $\Delta M = 3383 \text{ kNm}$, M = 3907 kNm whereon the testing stopped because the elastic rollback of the press was too large.

At this upload the deflection at the middle of the span measured 29.05 mm representing 1/78 of the opening and the maxim moment was $M=3907\ kNm$ representing 1.61 M $_{capULR}$.



Fig.1.Bridge beam L=24 m passage DN1



Fig.2.The distribution of the cracks



Fig.3 Evolution of the deflection

I.6.Conclusions

The behaviour of the prestressed beam L=24 m, PASSAGE DN1 was according to the technical requirements and conditions stated by STAS 12313-85 Railway bridges and main roads.

The test results showed a satisfactory behaviour of the beam, without any weaknesses and no adverse effects of the shear force, anchorage of the reinforcement, touching of the limits of the deformation in the reinforcement and concrete, attesting the manufacture manner of the beam in the technical manufacturers condition.

PART II

REINFORCED CONCRETE ROOF BEAM L=11.45M

II.1. General data

Beneficiary: SOMACO SA BUCURESTI Construction: INTEREX CENTRU COMERCIAL VASLUI

Special designer: PROCEMA Engineering SRL

Execution of the beam: SOMACO SA SUC. ROMAN

Beam testing: SC POPAESCU & CO srl București with the collaboration of SOMACO SA.

The test was effectuated on a stand of the SOMACO SA Branch ROMAN.

II.2. Data regarding the tested beam Materials:

• *Concrete* – the designer provided for Bc 25(C20/25), but at the execution there was used Bc 30(C25/30);

- concrete resistance after 24 hours was 28.5N/mm² and 46.4N/mm² after 28 days.

 $-R_{c} = 18 \text{ N/mm}^{2}, R_{t} = 1.25 \text{ N/mm}^{2}, Ec = 32500 \text{ N/mm}^{2}$

 Steel - for PC 52 the limit of flowing was between 336 şi 375N/mp, and the break resistance between 510 şi 579N/mp.

At the extremity of the beam was foreseen a horizontal notch (from shuttering) which permits for the beam to seat on the ather beam GP 18 AP.

By the structure of the beam can be noticed that the PC52- $2\Phi25$ steels are the only that had remainded at the inferior zone of the widening and

the others which are lifted on the widening, are insufficient anchored on the support of the beam, yet there are two horizontal steels PC 52 - Φ 16, which are anchorated with a ringlet.

The bending moment and the control shear force for the checkings at the vertical load –SLS, ULS (by the designer) at the middle of the opening:

- ULS maxim value M = 38.36 tfm, T=13.04 tf

- Exploitation –SLS M = 27.30 tfm, T = 9.25 tf.

II. 3. The test

There were foreseen two concentrated forces at 2.8m distance of the axle of the support. The stay of the beam was effectuated by two device support, one fixed and the another mobile.

The beam test was applied static with two 1200 kN hydraulic press, having the run over 250 mm, in points of load.

The weight of the ensemble press, the complementary plates was 1.0-1.2kN on each load poit.

II.3.1. Measurements

• There were effectuated measurements of apparition, opening of vertical and inclined cracks with magnifier glass with 0.1-0.02mm gradation and there were made diagrams of the cracks.

• There were effectuated measurements of arrows at the middle of the opening and at the poit of application of concentrated forces.

• Globally photos and in detail.

In continuation are presented the experimental values compareted with the supreme values.

II.4. Results and interpretation

 $M_r^{exp} = 64.4 \text{tfm}; M_{cap}^{ULS} = 45.06 \text{tfm},$

-relation $M_r^{exp} / M_{cap}^{ULS} = 1.429$ calculated for the $6\Phi 25 - PC$ 52 steel and concrete Bc 30, foreseen at the

execution, resulted 1.429>1.4. $T_r^{exp} = 23.0tf; \quad \tau_{cap}^{ULS} = 14.70 \text{ tf relation } T_r^{exp} / \tau_{cap}^{ULS}$

 T_r =25.00, T_{cap} =14.70 ft relation T_r = T_{cap} , calculated for 3 stirrup $\Phi 8$ and concrete Bc 30, foreseen at the execution, resulted 1.56.

The first cracks appears at the $0.9 P^{E}(1)$ step, there are marked in continuation at steps :9 tf(2), 12.7tf(3), 17.4tf(4), 20.2tf(5).

- at the (4) step was measured 0.35mm;

- at the (5) step the inclined cracks which starts from the inferior part of the widening, at the poit of interrupt of the 4 longitudinal steel, presented an opening over 2.0mm.

The beam was loaded in continuation till P=21 tf, were the deflection at the middle of the span measured 132.6mm, representing 1/84.4 of the opening , and the maxim moment M_r^{exp} represented 1.429 M_{cap}^{SLR} calculated for

 $6\Phi 25$ –PC 52 steel and Bc 30 concrete. There were no damages without the opening of the

inclined cracks over 0.2mm, which starts at the inferior part of the widening, in the point of interruption of the 4 longitudinal steel.

II.5. Conclusions

The behaviour of the reinforced concrete roof beam made by the SOMACO SA Branch ROMAN, conformable to Project PROCEMA Engineering SRL was according to the technical requirements and conditions specified by the standards şi technical regulations:

$$M_r^{\exp} / M_{cap}^{ULS} = 1.429 > 1.4.$$

 $T_r^{\exp} / T_{cap}^{ULS} = 1.56.$

The deflection at the middle of the span measured 132.6 mm, representing 1/84.4 of the opening. There was recomanded that at the design of similar beams to take carre of a better anchorage of the bars which are interrupted at the changing of the direction at the widening and of the inclined bars on support.



Fig.4. Extremity of beam where was foreseen a horizontal notch



Fig. 5. Tested beam



Fig. 6. Tested beam

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Fig. 7. Tested beam



Fig. 8. Tested beam

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Mitigation Strategies Used to Reduce the Effects of Natural Hazards

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Rezumat: Diminuarea dezastrelor este un domeniu ce cuprinde toate actiunile, toate masurile ce pot fi luate avand scopul de a reduce impactul unui dezastru inainte ca acesta sa se intample, inclusiv pregatirea si luarea masurilor de reducere a riscului pe termen lung. Include atat planificarea si implementarea masurilor de reducere a riscului, asociate cu hazarde naturale sau antropice, cat si procesul de planificare pentru masuri imediate ce pot fi luate in cazul aparitiei unui dezastru.Scopul general este acela de a sublinia conceptele de baza ale atenuarii dezastrelor si de a discuta sirul actiunilor de diminuare care pot fi considerate un raspuns la varietatea de hazarde naturale si antropice ce se pot ivi. Lucrarea prezinta conceptul de atentuare si face o scurta trecere in revista a anumitor hazarde care necesita o atentie sporita, descriind natura lor, consecintele pe care le pot avea si actiuni de diminuare a riscurilor in cazul fiecaruia.

Abstract: Disaster mitigation refers to all actions that have the purpose of reducing the impact of a disaster that can be taken prior to its occurrence, including preparedness and long-term risk reduction measures. It includes both the planning and implementation of measures to reduce the risks associated with known natural and humanmade hazards, and the process of planning for effective response to disasters which do occur. The general purpose is to outline the disaster mitigation concepts and to discuss the range of mitigation actions which can be considered as a response to the variety of natural and human-mad hazards which may be encountered. The paper debates the concept of mitigation and briefly surveys the range of hazards which may need to be considered, describing their nature, consequences and some of the mitigation actions specific to each.

Keywords: disaster, natural hazards, risk reduction, mitigation strategies, vulnerability.

1. General Considerations

Mitigation means taking actions to reduce the effects of a hazard before it occurs. The term mitigation applies to a wide range of activities and protection measures that might be instigated, from the physical, like constructing stronger buildings, to the procedural, like standard techniques for incorporating hazard assessment in land-use planning.

In the last two decades, as describe in [1] an substantial effort was made to encourage the implementation of disaster mitigation techniques in development projects all around the world. The United Nations has adopted the decade of the 1990s as the International Decade for Natural Disaster Reduction. The general aim is to achieve a significant reduction in the loss of life and material damage caused by disasters. Well trained organizations will play a central role in encouraging ISSN-1584-5990 national governments and non-governmental agencies to tackle disaster related issues through projects focused directly on reducing the impacts of hazards and through incorporation of risk awareness as part of the normal operations of development projects.

The great majority of casualties and disaster effects are suffered in developing countries. Development achievements can be wiped out by a major disaster and economic growth reversed. The promotion of disaster mitigation in the projects and planning activities of development protects development achievement and assists populations in protecting themselves against needless injury.

2. The General Concepts of Hazards, Vulnerability and Risk

Disasters occur when natural hazards have an impact on human beings. Those who have more resources – both economic as well as social – often © 2000 Ovidius University Press have a greater capacity to withstand the effect of a hazard than poorer members of a society. In developing countries, the root causes of vulnerability to hazards are poverty and inequitable development. Rapid population growth, urban or mass migration, inequitable patterns of land ownership, lack of education, and subsistence agriculture on marginal lands lead to vulnerable conditions such as unsafe siting of buildings and settlements, unsafe homes, deforestation, malnutrition, unemployment, underemployment, and illiteracy.

The figure below illustrates how the interface between vulnerable conditions and natural hazard can cause a disaster:



Fig. 1. Interdependantion between vulnerable conditions, natural hazards and disaster

Figure 1 depicts how increasing vulnerability increases the likelihood that a disaster will occur as a result of a hazard event. Since most hazard events are difficult (if not impossible) to control, one way of reducing disaster risk is to decrease the level of vulnerability.

Risk reduction is often perceived as restrictive, costly and incompatible with the community's economic development goals. In order to make progress toward adoption of risk reduction practices, local and national leaders need to recognize the constraints and barriers they face, secure the commitment of local communities, and develop innovative solutions for working with them.

The concepts of vulnerability, hazard, and risk are dynamically related. Community risk depends on the probability of occurrence and the magnitude of a hazard event, and how the particular hazard connects with the community's vulnerability. *Vulnerability.* The losses caused by a hazard, such as a storm or earthquake, will be proportionally much greater to more vulnerable populations – those living in poverty, with weak structures, and without adequate coping strategies. *Human vulnerability* is the relative lack of capacity of a person or community to anticipate, cope with, resist, and recover from the impact of a hazard. *Structural or physical vulnerability* is the extent to which a structure or service is likely to be damaged or disrupted by a hazard event. *Community vulnerability* exists when the *elements at risk* (defined below) are in the path or area of the hazard and susceptible to damage by it.

Hazard is defined as the potential occurrence, in a specific time period and geographic area, of a natural phenomenon that may adversely affect human life, property or activity to the extent of causing a disaster. The probability that a hazard will or will not occur, and its magnitude when it does occur also contributes to risk. Methods of predicting various hazards and the likelihood and frequency of occurrence vary widely by type of hazard.

Risk is defined differently by people in different situations. Risk as understood by a politician is different from risk to a seismologist, or to an insurance company executive, or to a family living in an earthquake zone. Risk is also different to local and national governments involved with disaster management.

For these policy makers, the *community elements at risk* include its structures, services, economic and social activities such as agriculture, commercial and service businesses, religious and professional associations and people. *Risk* is the expected losses to a community when a hazard event occurs, including lives lost, persons injured, property damaged and economic activities or livelihoods disrupted.

The relationship of these elements can be expressed as a simple mathematical formula which illustrates the concept that the greater the potential occurrence of a hazard and the more vulnerable a population, then the greater the risk:

$Risk = Hazard \times Vulnerability$ (1)

3. Know Your Enemy: Hazards & Their Effects

The most critical part of implementing mitigation is the full understanding of the nature of the threat. In each country and in each region, the types of hazards faced are different. Some countries are prone to floods, others have histories of tropical storm damage, and others are known to be in earthquake regions. Most countries are prone to some combination of the various hazards and all face the possibility of technological disasters as industrial development progresses. The effects these hazards are likely to have and the damage they are likely to cause depends on what is present in the region: the people, their houses, sources of livelihood and infrastructure. Each country is different. For any particular location or country it is critical to know the types of hazards likely to be encountered.

The understanding of natural hazards and the processes that cause them is the province of seismologists, volcanologists, climatologists, hydrologists and other scientists. The effects of natural hazards on structures and the man-made environment is the subject of studies by engineers and risk specialists. Death and injury caused by disasters and the consequences of damage in terms of the disruption to society and its impact on the economy is a research area for medical practitioners, economists and social scientists.

The science is still relatively young-most of the recordings of damaging earthquakes by strong motion instruments were obtained in the past twenty years, for example, and only since satellite photography has it been possible to routinely track tropical storms. The understanding of the consequences of failure of social organizations and regional economies is even more recent.

However there are now many books and case studies that document the incidence of disasters and a growing body of knowledge about hazards and their effects.

Understanding hazards involves comprehension of:

- \Rightarrow how hazards arise;
- \Rightarrow probability of occurrence and magnitude;
- \Rightarrow physical mechanisms of destruction;
- ⇒ the elements and activities that are most vulnerable to their effects;
- \Rightarrow consequences of damage.

Brief summaries of some of the major hazards and their effects are given in hazard-specific disaster mitigation summaries in the following pages.

4. Know Your Enemy: Vulnerability & Its Effects

Houses built from cane and thatches that can be blown apart in a tropical storm are more vulnerable to wind loads than a brick building. A brick building is more likely to disintegrate with the violent ground shaking of an earthquake than a strong reinforced concrete frame structure (or cane and thatch hut) and is more *vulnerable* to earthquake hazard. Vulnerability is the degree of expected damage form a particular hazard. Targeting mitigation efforts relies heavily on correctly assessing vulnerability.

This concept of vulnerability assessment can also be extended to social groups or economic sectors: people who rent their houses rely on a landlord to repair the damage and are more likely to be made homeless in the event of a disaster. Correctly identifying the groups of tenants and establishing rights of tenure and landlords' obligations to repair may reduce the number of people made homeless in a disaster. Similarly, food growers sending their produce to market through a single mountain pass will be unable to sell their produce if the pass is blocked. Developing an alternative route to market will reduce the vulnerability of the agricultural sector to damage by disaster.

5. Saving Life & Reducing Economic Disruption

The worst effects of any disaster are the deaths and injuries caused. The scale of disasters and the number of people they kill are the primary justifications for mitigation. Understanding the way that people are killed and injured in disasters is a prerequisite for reducing casualties. Among the sudden onset disasters, floods and earthquakes cause the most casualties worldwide, with storms and high winds being less deadly but far more widespread.

In earthquakes over 75% of fatalities are caused by building collapse. In floods deaths occur by drowning, mainly outdoors and in fast flowing currents or in turbulent water. Saving lives in earthquakes means focusing on prevention of building collapse. Reducing fatalities from floods means limiting the exposure of people to rapid inundation-either by keeping people out of the track of potential water flows or by preventing the flows from occurring.

The consequences of physical damage are often more important than the damage itself. A damaged factory can no longer continue to manufacture local economy suffers. Damage to infrastructure and to the means of production depresses the economy.

Mitigation also entails the protection of the economy from disasters. Economic activity in the more industrialized societies is complex and interdependent, with service industries dependent on manufacturing, which in turn relies on supplies of raw materials, labor, power and communications. This complex interdependency is extremely vulnerable to disruption by hazards affecting any one link in the chain. Newly industrializing societies are most vulnerable of all.

Agricultural sectors of the economy are most vulnerable to drought but also to floods and high winds, disease and pest attack and pollution. Industry is more vulnerable to earthquake damage and the disruption of transportation and utilities networks. Commerce and finance are most vulnerable to disruption of production, population migration and to breakdowns in communications systems. Mitigation measures that focus on protecting the most vulnerable elements and activities—the weakest links— in the different sectors of the economy will help protect the achievements of economic development.

6. Targeting Mitigation Where Has Most Effect

The understanding of how the occurrence of a natural hazard or an accident turns into a disaster enables us to forecast likely situations where disasters are possible. If there were no human settlements or economic activities affected, an earthquake would be a harmless act of nature. The combination of settlements *(elements)* and earthquake *(hazard)* makes the disaster possible.

Some elements are more *vulnerable* to earthquake effects than others. Identifying which these are (entitled *elements most at risk*) indicates priorities for mitigation.

Disasters are often the result of combinations of factors occurring together: a fire source, a dense residential area and combustible houses for example, or a seismic fault rupturing close to a city formed of high occupancy weak buildings. The contributory factors of past disasters can be identified to highlight similar conditions elsewhere. This is the process of risk analysis.

Identifying situations where combinations of risk factors coincide indicates the elements most at risk. The elements most at risk are the buildings, community services, infrastructure and activities that will suffer most from the effects of the hazard or will be least able to recover after the event. At a regional level, the concentrations of population and infrastructure in large cities make it likely that the losses inflicted by even low levels of hazard will exceed the total losses inflicted by severe levels of hazard on all the villages in the region. Mitigation measures in the city may have the most effect in reducing future losses. The portions of the housing stock in the city most likely to be damaged can be identified and mitigation measures applied to that sector will have the effect on reducing risk. The number of elements likely to be affected by a hazard, together with their vulnerability to the hazard will identify where mitigation is most effective.

7. Specific Hazards and Mitigation

Next, we will synthesize the particular characteristics of several hazard types and the main mitigation strategies used to reduce their effects, base on describe in [6] and our research:

7.1 Floods and Water Hazards

⇒ <u>Mechanism of destruction</u>

Inundation and flow of water with mechanical pressures of rapidly flowing water. Currents of moving or turbulent water can knock down and drown people and animals in relatively shallow depths. Debris carried by the water is also destructive and injurious. Structures are damaged by undermining of foundations and abutments. Mud, oil and other pollutants carried by the water is deposited and ruins crops and building contents. Flooding destroys sewerage systems, pollutes water supplies and may spread disease. Saturation of soils may cause landslides or ground failure.

⇒ <u>Parameters of severity</u>

Area flooded (km²), depth or height of flood, velocity of water flow, amount of mud deposited or held in suspension. Duration of inundation. Tsunamis or tidal waves measured in height (m).

⇒ <u>Causes</u>

River flooding results from abnormally high precipitation rates or rapid snow melt in catchment areas, bringing more water into the hydrological system than can be adequately drained within existing river channels. Sedimentation of river beds and deforestation of catchment areas can exacerbate conditions leading to floods. High tides may flood coastal areas, or seas be driven inland by windstorms. Extensive precipitation in urban areas or drainage failures may lead to flooding in towns as hard urban surfaces increase run-off loads. Tsunamis are caused by underwater earthquakes or eruptions. Dam failures or collapse of water retaining walls (sea walls, dikes, levees).

⇒ *Hazard assessment and mapping techniques*

Historical records give first indication of flood return periods and extent. Topographic mapping and height contouring around river systems, together with estimates of capacity of hydrology system and catchment area. Precipitation and snow-melt records to estimate probability of overload. Coastal areas: tidal records, storm frequency, topography and beach section characteristics. Bay, coastal geography and breakwater characteristics.

⇒ *Potential for reducing hazard*

Retaining walls and levees along rivers, sea walls along coasts may keep high water levels out of flood plains. Water regulation (slowing up the rate at which water is discharged from catchment areas) can be achieved through construction of reservoirs, increasing vegetation cover to slow down run-off, and building sluice systems. Dredging deeper river channels and constructing alternative drainage routes (new river channels, pipe systems) may prevent river overload. Storm drains in towns assist drainage rate. Beaches, dune belts, breakwaters also reduce power of tidal surges.

⇒ *Onset and warning*

Flooding may happen gradually, building up depth over several hours, or suddenly with the breach of retaining walls. Heavy prolonged precipitation may warn of coming river flood or urban drainage overload. High tides with high winds may indicate chance of coastal flooding some hours before it occurs. Evacuation may be possible with suitable monitoring and warning system in place. Tsunamis arrive hours or minutes after earthquake.

⇒ <u>Elements most at risk</u>

Anything sited in flood plains. Earth buildings or masonry with water-soluble mortar. Buildings with shallow foundations or weak resistance to lateral loads or impact. Basements or underground buildings. Utilities: sewerage, power, water supply. Machinery and electronics including industry and communications equipment. Food stocks. Cultural artifacts. Confined/ penned livestock and agriculture. Fishing boats and other maritime industries.

⇒ <u>Main mitigation strategies</u>

Land-use control and locations planning to avoid potential flood plain being the site of vulnerable elements. Engineering of structures in floodplain to withstand flood forces and design for elevated floor levels. Seepage-resistance infrastructure.

⇒ <u>Community participation</u>

Sedimentation clearance, dike construction. Awareness of flood plain. Houses constructed to be flood resistant (water-resistant materials, strong foundations). Farming practices to be floodcompatible. Awareness of deforestation. Living practices reflect awareness: storage and sleeping areas high off ground. Flood evacuation preparedness, boats and rescue equipment.

7.2 Earthquakes

⇒ <u>Mechanism of destruction</u>

Vibrational energy transmitted through the earth's surface from depth. Vibration causes damage and collapse of structures, which in turn may kill and injure occupants. Vibration may also cause landslides, liquifaction, rockfalls and other ground failures, damaging settlements in the vicinity. Vibration may also trigger multiple fires, industrial or transportation accidents and may trigger floods through failure of dams and other flood retaining embankments.

⇒ <u>Parameters of severity</u>

Magnitude scales (Richter, Seismic Moment) indicate the amount of energy release at the epicenter-the size of an area affected by an earthquake is roughly related to the amount of energy released. Intensity scales (Modified Mercalli, MSK) indicate severity of ground shaking at a location-severity of shaking is also related to magnitude of energy release, distance away from epicenter of the earthquake and local soil conditions.

⇒ <u>Causes</u>

Energy release by geophysical adjustments deep in the earth along faults formed in the earth's crust. Tectonic processes of continental drift. Local geomorphology shifts. Volcanic activity.

⇒ *Hazard assessment and mapping techniques*

Past occurrence of earthquakes and accurate logging of their size and effects: tendency for earthquakes to recur in the same areas over the centuries. Identification of seismic fault systems and seismic source regions. In rare cases it may be possible to identify individual causative faults. Quantification of probability of experiencing various strengths of ground motion at a site in terms of return period (average time between events) for an intensity.

⇒ *Potential for reducing hazard*: None.

⇒ *Onset and warning*

Sudden. Not possible to predict short-term earthquake occurrence with any accuracy.

⇒ <u>Elements most at risk</u>

Dense collections of weak buildings with high occupancy. Non-engineered buildings constructed by the householder: earth, rubble stone and unreinforced masonry buildings. Buildings with heavy roofs. Older structures with little lateral strength, poor quality buildings or buildings with construction defects. Tall buildings from distant earthquakes, and buildings built on loose soils. Structures sited on weak slopes. Infrastructure above ground or buried in deformable soils. Industrial and chemical plants also present secondary risks.

⇒ <u>Main mitigation strategies</u>

Engineering of structures to withstand vibration forces. Seismic building codes. Enforcement of compliance with building code requirements and encouragement of higher standards of construction quality. Construction of important public sector buildings to high standards of engineering design. Strengthening of important existing buildings known to be vulnerable. Location planning to reduce urban densities on geological areas known to amplify ground vibrations. Insurance. Seismic zonation and land-use regulations.

⇒ <u>Community participation</u>

Construction of earthquake-resistant buildings and desire to live in houses safe from seismic forces. Awareness of earthquake risk. Activities and day-today arrangements of building contents carried out bearing in mind possibility of ground shaking. Sources of naked flames, dangerous appliances etc. made stable and safe. Knowledge of what to do in the event of an earthquake occurrence; participation in earthquake practices, public drills, awareness programs. Community action groups for civil protection: firefighting and first aid training. Preparation of fire extinguishers, excavation tools and other civil protection equipment. Contingency plans for training family members at the family level.

7.3. Land Instabilities

⇒ <u>Mechanism of destruction</u>

Landslides destroy structures, roads, pipes and cables either by the ground moving out form beneath them or by burying them. Gradual ground movement causes tilted, unusable buildings. Cracks in the ground split foundations and rupture buried utilities. Sudden slope failures can take the ground out from under settlements and throw them down hillsides. *Rockfalls* cause destruction from fragmentation of exposed rock faces into boulders that roll down and collide into structures and settlements. *Debris flows* in softer soils, slurry material, man-made spoil heaps and soils with high water content flow like a liquid, filling valleys, burying settlements, blocking rivers (possibly causing

floods) and blocking roads. *Liquefaction* of soils on flat land under strong vibrations in earthquakes is the sudden loss of the strength of soils to support structures that stand on it. Soils effectively turn temporarily to liquid allowing structures to sink *Parameters of severity*

Volume of material dislodged (m3), area buried or affected, velocity (cm/day), boulder sizes. ⇒ *Causes*

Gravitational forces imposed on sloping soils exceed the shear strength of soils that hold them in position. High water content makes soil heavier, increasing the load, and decreasing shear strength. With these conditions heavy rainfalls or flooding make landslides more likely to happen. The angle of slope at which soils are stable is a physical property of the soil. Steep cuttings through some types of soils makes them unstable. Triggering of the collapse of unstable soils can be caused by almost any minor event: storms, minor ground tremors or man-made actions. Liquifaction is caused by earthquake vibrations through loose

⇒ *Hazard assessment and mapping techniques*

Identification of previous landslides or ground failures by geotechnical survey. Identification of probability of triggering events such as earthquakes. Mapping of soil types (surface geology) and slope angles (topographic contouring). Mapping of water tables, hydrology and drainage. Identification of artificial land fill, man-made mounds, garbage pits, slag heaps. Investigation into the probability of triggering events, especially earthquakes.

⇒ <u>Potential for reducing hazard</u>

soils, usually with high water content.

Landslide risk for a slope reduced by shallower slope angles (excavating top layer to cut back slope), increasing drainage (both deep drainage and surface run-off) and engineering works (pilling, ground anchors, retaining walls). Shallower angles for embankments and cuttings, terracing slopes and forestation can prevent loss of surface material to depth of root penetration. Debris flows can be directed into specially constructed channels if they are expected. Rockfall protection barriers (trenches, slit dams, vegetation barriers) can protect settlements.

⇒ *Onset and warning*

Most landslides occur gradually at rates of a few centimeters an hour. Sudden failures can occur

without warning. Rockfalls are sudden but noisy. Debris flows sudden, but precursory trickles of material may give a few minutes of warning if population is prepared.

\Rightarrow <u>Elements most at risk</u>

Settlements built on steep slopes and softer soils or along cliff tops. Settlements built at the base of steep slopes, on alluvial outwash fans or at the mouth of streams emerging from mountain valleys. Roads and other communication lines through mountain areas. Masonry buildings. Buildings with weak foundations. Large structures without monolithic foundations. Buried utilities, brittle pipes.

⇒ <u>Main mitigation strategies</u>

Location planning to avoid hazardous areas being used for settlements or as sites for important structures. In come cases relocation may be considered. Reduce hazards where possible. Engineering of structures to withstand or accommodate potential ground movement. Piled foundations to protect against Liquefaction. Monolithic foundations to avoid differential settlements. Flexible buried utilities. Relocation of existing settlements or infrastructure may be considered.

⇒ <u>Community participation</u>

Recognizing land instability potential and identifying active landslides. Avoidance of sitting houses in hazardous locations. Construction of strong foundations for structures. Compaction of ground locally. Slope stabilization through terracing and forestra. Rockfall barriers (trees and earth banking).

7.4 Strong winds (typhoons, hurricanes, cyclones, tropical storms and tornados)

⇒ <u>Mechanism of destruction</u>

Pressure and suction from wind pressure, buffeting for hours at a time. Strong wind loads imposed on a structure may cause it to collapse, particularly after many cycles of load reversals. More common damage is building and non-structural elements (roof sheets, cladding, chimneys) blown loose. Wind-borne debris causes damage and injury. High winds cause stormy seas that can sink ships and pound shorelines. Many storms bring heavy rains. Extreme low air pressure at the center of a tornado is very destructive and houses may explode on contact.

⇒ <u>Parameters of severity</u>

Velocity of wind. Wind scales (e.g. Beaufort) gale severity scale. Local hurricane/typhoon scales.

⇒ <u>Causes</u>

Winds generated by pressure differences in weather systems. Strongest winds generated in tropics around severe low pressure systems several hundreds of kilometers diameter (cyclones) known as typhoons in the Pacific and as hurricanes in Americas and elsewhere. Extreme low pressure pockets of much narrower diameter generate rapidly twisting winds in tornados.

⇒ <u>Hazard assessment and mapping techniques</u>

Meteorological records of wind speeds and direction at weather stations gives probability of high winds in any region. Local factors of topography, vegetation and urbanization may affect microclimate. Past records of cyclone and tornado paths give common patterns of occurrence for damaging wind systems.

⇒ <u>Potential for reducing hazard</u>

None. Cloud seeding may dissipate rain content. ⇒ <u>Onset and warning</u>

Tornados may strike suddenly but most strong winds build up strength over a number of hours. Low pressure systems and tropical storm development can be detected hours or days before damaging winds affect populations. Satellite tracking can help follow move ment of tropical storms and project likely path. The movements of weather systems are however, complex and still difficult to predict with accuracy.

⇒ <u>Elements most at risk</u>

Lightweight structures and timber housing. Informal housing sectors and shanty settlements. Roofs and cladding. Loose or poorly attached building elements, sheets and boards. Trees, fences, signs etc. Telegraph poles, pylons and high-level cables. Fishing boats or other maritime industries.

⇒ <u>Main mitigation strategies</u>

Engineering of structures to withstand wind forces. Wind load requirements in building codes. Wind safety requirements for non-structural elements. Good construction practices. Microclimatic siting of key facilities, e.g. in lee of hillsides. Planting of windbreaks, planning of forestry areas upwind of towns. Provision of windsafety buildings (e.g. strong village halls) for community shelter in vulnerable settlements.

Construction of wind-resistant or easily rebuilt houses. Securing fixing of elements that could blow away and cause damage or injury elsewhere, e.g. metal sheeting, fences, signs. Preparedness for storm action. Taking shelter in strong, wind-resistant buildings. Protection measures for boats, building contents or other possessions.

8. Conclusion

This paper is a general insight initiation into the concept of disaster mitigation and provides specific mitigation information for several major hazard types. It also endows you with the usage of mitigation activities so that they can be applied with their best efficiency.

These demonstrate that hazards have different effects on different parts of the community, sectors of the economy and types of infrastructure: floods tend to destroy agricultural produce but cause less damage to the structure of buildings; earthquakes tend to destroy structures but have little impact on crops growing in fields. The *vulnerability* of people, buildings, roads, bridges, pipelines, communications systems and other elements is different for each hazard.

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Wave forces related to waves conditions and structures characteristics

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Rezumat: Valurile marine au forme geometrice neregulate și amplitudini variabile. Calculul forțelor pe care le produc asupra construcțiilor hidrotehnice maritime este dificil și aproximativ. În lucrare se prezintă succint modul de evaluare a forțelor din valuri funcție de caracteristicile impactului dintre val și structură și se propune un procedeu de calcul care permite o analiză rapidă și simplă a acțiunii valurilor sub forma unor spectre dinamice de răspuns. Spectrele dinamice de răspuns se obțin prin rezolvarea numerică a unei integrale de tip Duhamel. Pe baza acestor spectre, se poate calcula apoi răspunsul dinamic maxim, modelând structura ca un sistem echivalent cu un singur gard de libertate sau cu mai multe grade de libertate dinamică. Lucrarea conține exemple numerice rezolvate.

Abstract: Sea waves have irregular geometrical shapes and varying amplitudes. It is very difficult to make an approximation of the forces they produce on the maritime structures. Wave-generated pressures on structures related to the wave conditions and geometry of the structure is presented in this work. Within this work, a calculation procedure is suggested, a procedure that allows a quick and simple analysis of the sea waves loadings under the form of dynamic response spectrums. These can be estimated using the registered oscillogrames of the sea waves, that become oscillogrames of the wave pressure on the maritime structure. The dynamic response spectrums are estimated by solving numerically an integral of Duhamel type. Taking as basis these spectrums, the maximal dynamic response could be calculated, modeling the structure as an equivalent system with a single or multiple degrees of freedom. This work contains the solutions of some numerical examples.

Keywords: wave forces, pressure distribution, arbitrary general dynamic loading, maximal dynamic response

1. Introducere

The wave forces represent an important element in designing the maritime structures, being, at the same time, hard to be calculated and subjected to several uncertainties.

Most of the times, the sea waves are considered, in a simplified way, as regulated waves, having a quite sinusoidal shape, named monochromatic waves (Fig.1a). The visual observations and the measurements show that the sea surface is formed of waves having variable periods and heights and various directions. The term "irregular waves" is used to design the natural state of the sea for which the waves characteristics present statistic variability, as opposed to the monochromatic waves, the properties of which are considered to be constant. In the recordings made in

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fixed points, there could be noticed non-repetitive wave profiles (Fig.2).



Fig. 1Sea waves representations

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Fig. 2 An irregular wave profile, resulted from records

Fig. 3 indicates in a simplified way, the dynamic character of the sea wave's loadings, by

Even if there could be noticed also individual waves, these ones present significant individual variations of the height and of the period, from one wave to another. The recordings from a certain point represent the superposition of several waves, of different directions, that are intercrossed at one specific moment. The sea state during the storm is always well-represented by the irregular waves (short-crest waves). The long-crest waves (or the monochromatic waves) are suited to describe the swell-state of the sea, when the waves characteristics vary in a restraint range.

changing the intensity, direction and vertical position during the wave cycle.



Fig. 3 The dynamic character of the horizontal forces generated by waves

Wave-generated pressures on structures are complicated functions of the wave conditions and geometry of the structure. Fig. 4 show several types of time-varying wave impact loading. Even if the waves' action has a dynamic character, taking into consideration that the fundamental period of the defending structures (0,2...2s) is significantly smaller than the period of the regular waves, the tension state is deduced usually by the static application of the wave loading.

We present hereafter the usual way of determining the value of the pressures due to the waves, for the structures with vertical walls and for rubble mound structures.

2. The estimation of the pressure force of the waves on the structures with vertical walls

The horizontal force induced by the action of the waves is deduced according to the effect that the waves have on the vertical walls.

The total hydrodynamic pressure distribution on a vertical wall consists of two time-varying components: the hydrostatic pressure component due to instantaneous water depth at the wall, and the dynamic pressure component due to acceleration of the water particles.

There can be identified three types of timevarying force generated by the wave on the vertical walls, that are summarized in Fig. 5. For nonbreaking waves, the pressure at the wall has a slight variation in time. This load is called pulsating or quasi-static because the period is much longer than natural period oscillation of the structure. Consequently the wave load can be treated like a static load. A special consideration is required if the wall is placed on fine soils where pore pressure may build up, resulting in significant weakening of the soil.

Sainflou's formula, derived theoretically for regular, non-breaking waves (but applied also for irregular waves) assumes a linear pressure distribution below the water level (Fig.6). This relation cannot be used for breaking or overtopping waves.

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Fig. 4 [A. Kortenhaus et.a.]

Waves that break in a plunging mode develop an almost vertical front before they curl (Fig. 5b). If this almost vertical front occurs just prior to the contact with the wall, then a very high pressure is generated. This pressure has extremely short durations. A negligible amount of air is entrapped and result a very large single peak force followed by very small force oscillations). The duration of the pressure peak is of the order of hundredths of a second. Goda's formula assumes trapezoidal shape for pressure distribution along front. (Fig. 7)

If a large amount of air is entrapped in a pocket, a double peaked force is generated followed

by pronounced force oscillations (Fig. 5c). The first and largest peak is induced by the wave crest impact on the structure. The second peak is induced by the subsequent maximum compression of air pocket. The force oscillations are due to the pulsation of air pocket. The double peaks have spacing in range of milliseconds to hundredths of seconds, and period of the force oscillations is in the range 0.2-1seconds.(Fig. 8)

Based on the registration of the pressure shocks, Minikin's method leads to overestimated pressures, (15 to 18 times higher), having their maximal level at or near the still water level.



Fig. 5 Three type of wave forces on verical wall: a) non-breaking wave, b) – breaking wave with almost vertical front c) – breaking wave with large air pocket



Fig. 6 Pressure distribution within the wave (regardless to the hydrostatic pressure) for the non-breaking waves



Fig. 7 The pressure distribution within the wave for the breaking waves (Goda)



Fig. 8 The estimation and the distribution of the pressures by Minikin method for the cases in which the breaking wave's crest is bellow the design breaker crest

3 Calculation of the pressure force within the waves against the rubble mound structures

Due to the complex movement of the water during the breaking of the waves on the rubble mound, it was not possible to evaluate, exclusively by theoretical methods, a formula for the force that the waves deploy over the protection structures. It starts, conventionally, from the hypothesis that, after the breaking, the movement is produced following a parabolic trajectory, noted as AB in Fig. .9

Starting from the components of velocity on the crest of the wave $(V_x=V_A \text{ and } V_y=0)$ and from the equation of the rubble mound (y=x/m), resulting the coordinates of the impact point, B, that depends mostly on the velocity at the wave crest level.



Fig. 9 The waves' impact on a rubble mound

In order to allot the pressures exercised by the wave, figured in Fig. 10, there are empirical relations

that are applied, in which interfere the impact velocity following the normal direction against the rubble mound, V_B , which is obtained also by applying semi-empirical relations.

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Fig. 10 The distribution of the pressures on the rubber mound [3]

Another formulation, very similar, is given by the Djunkovski [4] method, and the Iribarren method uses a simplified formula for the maximal pressure in the impact point, without taking in consideration its dependency on the slope.

As opposed to the usual way of dealing with the loadings within the wave, as forces that are applied statically, in this work the waves are considered as some certain dynamic loads and it is proposed a simplified formula of calculation for the action given by the waves, based upon the concept of the response spectrum. The spectral response forms the most convenient means for to illustrate the maximal response. There will be calculated the spectrums of the displacements, velocities and accelerations. Each of the three spectrums gives a certain quantity, with a different physical significance. The displacement spectrum indicates the maximal displacement of the system, the velocity spectrum indicates the maximal specific energy stored within the system, and the acceleration spectrum refers to the maximal value of the equivalent static force.

4 The calculation procedure

Sea wave loading is considered as **arbitrary general dynamic loading** p(t), specifically the intensity of loading $p(\tau)$ acting at time $t=\tau$. The entire loading history may be considered to consist of a succession of short impulses $p(\tau)d\tau$, each producing its own differential response. For linearly elastic system, the total response can be obtained by summing all the differential response, using Duhamel Integral. Concerning a natural frequency ω , and the critical damping coefficient, v, the instantaneous displacement is estimated following the formula:

$$u(t) = \frac{1}{m \cdot \omega^*} \int_0^t p(\tau) e^{-v\omega(t-\tau)} \sin \omega^*(t-\tau) \,\mathrm{d}\,\tau \tag{1}$$

where $\omega^* = \omega \sqrt{1 - v^2}$ represents the own pulsation when damping is present [1].

Considering the trigonometrically identity $\sin(\omega t - \omega \tau) = \sin \omega t \cos \omega \tau - \cos \omega t \sin \omega \tau$, the response in time could be obtained by the following formula:

$$u(t) = \frac{1}{m \cdot \omega^*} \frac{\sin \omega t}{e^{v\omega t}} \int_0^t p(\tau) e^{v\omega \tau} \cos \omega^* \tau \, \mathrm{d} \tau - \frac{1}{m \cdot \omega^*} \frac{\cos \omega^* t}{e^{v\omega t}} \int_0^t p(\tau) e^{v\omega \tau} \sin \omega^* \tau \, \mathrm{d} \tau =$$
(2)
$$= \frac{1}{m \cdot \omega^*} \frac{\sin \omega^* t}{e^{v\omega t}} A(t) - \frac{1}{m \cdot \omega^*} \frac{\cos \omega^* t}{e^{v\omega t}} B(t)$$

where the expression of A(t) and B(t) integrals

$$A(t) = \int_{0}^{t} p(\tau) e^{v\omega\tau} \cos \omega^{*} \tau \,\mathrm{d}\,\tau$$

$$B(t) = \int_{0}^{t} p(\tau) e^{v\omega\tau} \sin \omega^{*} \tau \,\mathrm{d}\,\tau$$
(3)

can be obtained by means of one of the integration numerical methods. By deriving the Eq. (2), there were obtained the formulae for the velocity and reaction acceleration, having the form of the following formulae:

$$u(t) = \frac{\cos \omega t}{m \cdot e^{v\omega t}} A(t) + \frac{\sin \omega t}{m \cdot e^{v\omega t}} B(t) - \omega v u(t)$$
(4)

$$u(t) = -u(t)\omega^2 - 2u(t)\omega\nu$$
⁽⁵⁾

Deducing the response for a structure that has an elastic behavior, with a single degree of freedom, represents the main part of RS-WAVE code, written in

5 Numerical examples

the language FORTRAN 90. The formulae of the Eq. (3) were deduced by using the integration rule of Simpsons.

The RS-WAVE code gives the necessary data for to represent the response spectrums, but also those needed in order to represent the response history for a structure of which the own period is known, under the conditions in which the general dynamic actions are directly applied to the oscillating system.

The RS-WAVE code requests the following entry data:

- the oscillograme, described either by the variation of the recorded forces, estimated according to time, or by variation of the wave surface profile;
- the mass of the oscillating system and, if necessary, the own pulsation and the coefficient from the critical amortization.

There could also be produced some oscillogrames of the variation of the wave surface profile by the wave recorders, and other oscillogrames of the forces could be obtained by means of the tension traductors.

If the processed data consist of oscillogrames of the variation of the wave surface profile, the force that is directly applied to the oscillating system is estimated according to the position of the element towards the still water level.



Fig. 11 Representation of h1 and h2 heights as required by RS-WAVE code

Figure 12a shows the response spectra of displacement, velocity and acceleration calculated for oscillogrames of the variation for the wave surface profil.

Figure 12b shows the response spectra of displacement, velocity and acceleration calculated for oscillogrames of pressure which could be obtained by means of the tension traductors. The first pressure is 60kPa followed at 150 milisecoonds by second peak pressure of 120kPa, by [5].

6 Results

This paper treats wave loading as a arbitrary general dynamic loading. From the point of view of the structure engineer, the maximal result of these dynamic actions applied directly to the oscillating system could be easily obtained, using dynamic response spectrums.

The wave profile oscillograme becomes the oscillograme of the hydrostatic pressure given by the wave on the structure. By solving numerically the Duhamel integral, the necessary data for a representation of the maximal response under the form of the spectrums can be obtained.

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Fig. 12 Response spectra of displacement, velocity and acceleration for: (a) m=1000t , oscillogram for wave surface, h_1 =4m, h_2 =4.2m; (b) m=1000t, oscillogram for pressure recorded by tension traductor
Measurement and estimation of the longshore sediment transport Boumerdes coast, Algeria

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Rezumat: Intelegerea mecanismului de transport al sedimentelor este o componenta importanta in managementul zonei costiere. Curgerea sedimentelor modeleaza sistemul de dune, plaje si bancuri offshore care sunt cruciale pentru protectia coastei si frumusetea plajei. Estimarile ratelor de transport (longshore) ale sedimentelor in lungul tarmului, in zona de studiu, sunt necesare pentru o evaluare globala a regimului de sedimentare si pot furniza o viziune a potențialelor schimbari pe termen lung a pozitiei liniei tarmului si stocului de sedimente. Un numar de formule empirice si semi-empirice de transport al sedimentelor au fost dezvoltate pentru a fi utilizate în aplicațiile costiere. Scopul acestei lucrari este de a compara patru dintre aceste formule: Bijker, Kamphius, Van Rijn și CERC. Totalul de LSTR dea lungul zonei de surf a fost calculat folosind formula Kamphius (care a dat cele mai bune rezultate) dupa ce a fost introdus coeficientul de corelare.

Abstract: Understanding of coastal sediment transport is an important component in coastal zone management. Sediment flows shape the system of dunes, beaches and offshore banks that are crucial for coastal protection and beach amenity. Estimates of longshore sediment transport rates within the study area are required for an assessment of the overall sedimentation regime and can provide an insight into potential long-term changes in the shoreline position and sediment budget. A number of empirical and semi-empirical sediment transport formulae have been developed for use in coastal applications. The aim of this paper is to compare four of these formulae: the Bijker, Kamphius, Van Rijn and CERC formulae. The total LSTR along the surf zone was calculated using Kamphius formula (that has given the best results) after introducing the coefficient of correlation.

Keywords: Boumerdes, longshore, sediment transport.

1. Introduction

Natural beaches change constantly in response to natural conditions (i.e. wind and wave forcing) or human interference (e.g. breakwater, groin construction). Modelling and prediction of such bathymetric changes requires an accurate knowledge of sediment transport processes in the nearshore and in particular within the surf zone. These processes are controlled by the physical mechanisms of sediment mobilisation, as it is influenced by the prevailing hydrodynamic and

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bottom boundary layer conditions, by the coupling of sediment and fluid motion (sediment fluxes) and their variability in space and time.

The alongshore and cross-shore components of the water motion at the breaking process of obliquely approaching waves cause cross-shore and longshore currents which will also move sediment in the region. There are two mechanisms beach drifting in the swash zone and transport in the breaking zone (Kamphuis, 2000).

A number of empirical and semi-empirical sediment transport formulae have been developed for

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use in coastal applications. The objective of the present study is to evaluate the predictive capability of four well-known sediment transport formulas, adapted to calculate cross-shore distribution of the longshore sediment transport rate (LSTR), based upon an experimental field data set. We selected formulas that we have gained world-wide acceptance in confidently predicting longshore sediment transport rates.

2. Transport processes

Wave motion over an erodible sand bed can generate a suspension with large near-bed concentrations, as shown by laboratory and field measurements. Mean currents such as tide- windand density-driven currents carry the sediments in the direction of the main flow; this type of transport usually is termed the current-related transport.

Wave-induced transport processes are related to the oscillating and mean currents generated in the wave boundary layer by high-frequency waves. Net onshore transport due to wave asymmetry generally is dominant in non-breaking wave conditions outside the surf zone, whereas net offshore transport generally is dominant during conditions with breaking waves.

The major transport components contributing to the wave-induced transport processes are:

- Net onshore-directed transport (bed load and intermittent suspended load) due to asymmetry of the near-bed orbital velocities with relatively large onshore peak velocities under the wave crests and relatively small offshore peak velocities under the wave troughs.

- Longshore-directed transport due to the generation of longshore wave-driven currents due to breaking waves.

- Net offshore-directed transport due to the generation of a net return current (undertow) in the near-bed layers balancing the onshore mass flux between the crest and trough of breaking waves.

- Net onshore-directed transport due to the generation of a quasi-steady weak current (Longuet-Higgins, 1970) in the wave boundary layer.

- Net offshore-directed transport due to the generation of bound long waves associated with variations of the radiation stresses under irregular

wave groups (peak velocities and sand concentrations are out of phase).

- Gravity-induced transport components related to bed slopes.

3. The sediment transport formulae studied

We chose to study four formulas which are interesting because of their different approaches to the problem:

- **The CERC formula** (Coastal Engineering Research Center for predicting the total rate of longshore transport) (Shore Protection Manual ,1984):

$$Q = KA \frac{\rho g^2}{64 \pi} TH_b^2 \sin 2\alpha_b \tag{1}$$

Where Q = volume of longshore transport rate in m³/year, K = dimensionless constant relating sand transport to longshore energy flux and was taken as 0.39, A= 1/[(ρ_s - ρ)g(1-p)], ρ_s = mass density of the sediment (2650 kg/m³), ρ =mass density of seawater (1025 kg/m³), g = acceleration due to gravity (9.81m/s²), p = porosity of sediment (0.4), T = wave period in s, H_b = breaking wave height in m, and α_b = breaker angle with respect to coastline.

- **The Bijker formula**: One of the first sediment transport formulations that is still often used in engineering applications was proposed by (Bijker, 1968). It is derived from (Frijlink, 1952) formula for a current only with a modification of the bottom shear stress using a wave-current model. The direction of sediment fluxes is always that of the current since this formula was proposed to estimate longshore transport rate:

$$\begin{cases} q_{sb} = C_b d \sqrt{\frac{\mu_c \tau_c}{\rho}} \exp\left(-027 \frac{(\rho_s - \rho)}{\mu_c \tau_{cw}}\right) \\ q_{ss} = 1.83 q_{sb} \left(I_1 \ln\left[\frac{33h}{\delta_c}\right] + I_2\right) \end{cases}$$
(2)

Where q_{sb} , q_{ss} : sediment volume fluxes for bed load and suspended load, respectively; *d*: median grain size diameter; *h*: water depth; C_b : breaking wave parameter; μ_c : ripple parameter; τ_c : shear stress due to current only; τ_{cw} : shear stress due to wave–current interaction; ρ_s , ρ : sediment and water densities, respectively; I_1 , I_2 : Einstein integrals (suspended load); and $\delta_c = 100d / h$: dimensionless thickness of the bed load layer.

The ripple parameter introduced by (Bijker, 1968) is defined by the following equation:

$$\mu_c = \left(\frac{f_{ct}}{f_c}\right)^{3/2} \tag{3}$$

Where f_{ct} : the total friction coefficient due to current and f_c : the skin friction coefficient due to current.

The breaking wave coefficient is defined by:

$$C_{b} = \begin{cases} 2; if : H_{w}/h < 0.05\\ 2+3(H_{w}/h-0.05); if :< 0.05H_{w}/h < 0.4 \\ 5; if : 0.4 < H_{w}/h \end{cases}$$
(4)

Where H_w : wave height and h: water depth.

The shear stress due to the wave–current interaction is computed following the method proposed by Bijker introducing a suspension factor:

$$\tau_{cw} = \left[1 + 0.5 \left(\xi_B \frac{U_w}{U_c}\right)^2\right] \tau_{cf}$$
(5)

With $\xi_B = \sqrt{f_{wt} / f_{ct}}$: parameter due to the wave-current interaction, f_{wt} : the total friction coefficient due to waves, U_w : peak value of the wave orbital velocity at the bottom, and U_c : mean current velocity.

-The Kamphuis formula: (Kamphuis, 1991) developed a relationship for estimating longshore sediment transport rates based primarily on physical model experiments. The formula which (Kamphuis, 2002) found to be applicable to both field and laboratory data is:

$$Q_{lst,m} = 2.27 H_{s,b}^2 T_P^{1.5} m_b^{0.75} D_{50}^{-0.25} \sin^{0.6}(2\theta_b) \qquad (6)$$

in which $Q_{lst,m}$ is the transport rate of immersed mass per unit time, T_p is peak wave period, m_b is the beach slope near the breaking, i.e., the slope over one or two wavelengths seaward of the breaker line, and D_{50} is the median grain size. The immersed weight is related to the volumetric rate as $Q_{lst,m} = (\rho_s - \rho)(1-a) Q_{lst,}$. The Kamphuis formula is appealing since it includes the wave period and beach slope, which both influences wave breaking, and the grain size, which should be an important factor for the mobilization and transport of sediment.

- The Van Rijn formula: The (Van Rijn,1989) formula is expressed in the same way as the Bijker formula, as the sum of bed load transport (taking into account the influence of waves) and the suspended load flux integrated over depth. The direction of sediment fluxes is also that of the current. Bed load transport can be written as follows:

$$q_{sb} = 0.25 dD_*^{-0.3} \left(\frac{\tau_{cf}}{\rho}\right)^{0.5} \left(\frac{\tau_{cw} - \tau_{cr}}{\tau_{cr}}\right)^{1.5}$$
(7)

Where $D_*=((s-1)gd_{50} / v^2)^{1/3}$: dimensionless sediment diameter, $\tau_{cf} = \mu_c \alpha_{cw} \tau_c$: total shear stress due to current only (taking into account the influence of bed forms), τ_{cr} : critical shear stress for sediment transport, $\mu_c = f_c/f_{ct}$: shape factor, and α_{cw} : coefficient due to the presence of waves (which can affect the mean shear stress).

Suspended load transport is computed by solving the equation of concentration over depth:

$$\frac{dc}{dz} = -\frac{(1-c)^5 c W_s}{\varepsilon_{scw}}$$

Where c(z): mean volume concentration (time averaged) at height z, $(1 - c)^5$ corresponds to the decrease of the settling velocity due to high concentrations, and ε_{scw} : mixing coefficient in case of a wave–current interaction.

Then, integrating sediment fluxes over depth:

$$q_{ss} = \int_{a}^{b} \overline{u}(z)c(z)dz$$
(8)

Where h: water depth; $\alpha = \max(k_{sct}, k_{swt})$: reference level; k_{sct} , k_{swt} : total roughness values due to current and waves; and $\overline{u}(z)$: mean velocity (time averaged) at height z.

 ε_{scw} , c_a and $\overline{u}(z)$ are computed following the equations given by (Van Rijn, 1984, 1989).

The depth averaged current was calculated from the following Eq. (3) and used as input along with the calculated d_{50} and d_{90} size of the sediments collected at traps.

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$$\overline{u}(z) = \begin{cases} \left(\frac{z}{0.32h}\right)^{1/7} u; \text{ for } : 0 < z < 0.5h \\ 1.07u; \text{ for } : 0.5h < z < h \end{cases}$$
(9)

4. Study area

A 1 km-long segment of beach at Boumerdes (Algeria) (Fig. 1), on the east coast of Algiers, was selected for the study. The beach is straight and open. Boumerdes Beach is situated between Boumerdes River from the East and Tatareg River in the west.

The site under study is located on the following geographical coordinates: $(3^{\circ}24' - 3^{\circ}29')$ East and $(36^{\circ}46'-36^{\circ}50')$ North.

4.1. Hydrodynamic and sediment characteristics

The coastline of the studied area consists of sandy beaches with sediment sizes in the range of 100 to 2000 μ m with a predominant grain size from 110 to 200 μ m.



Figure 1 Geographical situation of the study area

Most important for the coastal morphology is the highly variable wave climate near the coast.

The wave climate at the breaker zone is dominated by waves of a moderate height (about 1.2 m) and a relatively short period (about 6 s). Waves offshore exceed 2 m approximately 4.39% of the time and 3 m approximately1.07% of the time. Most waves arrive from the northwest (300°) to the southeast and from northeast to the southwest. The highest waves are from the west direction because of the longer fetches in this sector. Swell is also dominant from the northeast direction.

The average coastal orientation of the coast is 90° clockwise to north. The longshore current direction towards 90° is mentioned as the flow towards East, and that towards 270° is mentioned as West in this study. Variation of longshore current velocity is calculated from the Berthois formula (1969):

$$V = K\sqrt{gH_b / Ti\sin 2\alpha_b} \tag{10}$$

Where K is a coefficient that depends on sea bed roughness, it's usually taken 2.6.

5. Results and discussions

5.1. Measured LSTR

The measured LSTR in the surf zone varied from: $50*10^{-6}$ to $430*10^{-6}$ m³/s, with an average value of $187*10^{-6}$ m³/s. The measured distribution is shown in Fig 2; it shows that the LSTR was high in the direction 290°.

5.2. Calculated LSTR

The wave measurements transformed to the breaker zone were input to the CERC (Eq. (1)), and the calculated values are presented in Fig. 2. The longshore sediment varied from: $-48*10^{-6}$ m³/s to $286*10^{-6}$ m³/s with a maximum value in the direction 260° .

The LSTR was also calculated using Camphius, Bijker and Van Rijn equations, and presented in Fig. 2. The LSTR varied from: $-511 \text{ m}^3/\text{s}$ to $129*10^{-6} \text{ m}^3/\text{s}$ (with Camphius), $-511*10^{-6}$ to $127*10^{-6} \text{ m}^3/\text{s}$ (with Bijker) and from $-821*10^{-6}$ to $79*10^{-6} \text{ m}^3/\text{s}$ (with Van Rijn).



Figure 2 Variation of LSTR (measured and calculated) with dominant wave direction

5.3. Comparison of measured and calculated LSTR

As a measure of scatter, the RMS error (σ_{rms}) was calculated according to:

$$\sigma_{rms} = \left[\sum_{1}^{n} \left[\log(Q_{c}) - \log(Q_{m})\right]^{2} / (n-1)\right]^{1/2} \quad (11)$$

Where *n* is the number of data points, $Q_c(Q_m)$: are respectively the calculated (measured) value.

The correlation coefficient (r) between the measured and calculated LSTRs was also calculated with:

$$r = \frac{(\sum_{1}^{n} Q_{c} . Q_{m}) - n . \overline{Q}_{cd} . \overline{Q}_{m}}{[(\sum_{1}^{n} Q_{c}^{2} - n \overline{Q}_{c}^{2})(\sum_{1}^{n} Q_{m}^{2} - n \overline{Q}_{m}^{2})^{0.5}]^{0.5}}$$
(12)

For a perfect matching, the correlation coefficient value will be unity.

The RMS error between the measured and that calculated based on CERC formula was 0.38 and the correlation coefficient (r) calculated was 0.86. The smaller RMS error implies smaller scatter. (Kamphuis et al.1986) found reasonable results with the CERC formula for particle size in the range of 0.2 to 0.6 mm. In the present case, the median size varied from 0.10 to 0.20 mm, but the deviation between the measured and that predicted by CERC was acceptable.

Based on the measured total LSTR by the streamer traps and short term impoundment along the low-wave energy coasts, (Wang et al.,1998) found that the rates measured were lower than that predicted by the various empirical formulas. Using the root mean square wave height in the CERC formula, the empirical coefficient K was found to be 0.08 instead of 0.78 recommended in the Shore Protection Manual. For an average wave height of 0.4 m, (Wang et al., 1998) found that the values calculated using CERC formula is nine times greater than the trap-measured values. Because the average wave height in the present study was double the value observed by (Wang et al., 1998), a better correlation was found between the measured and calculated values.

The RMS error between the measured and that calculated based on Kamphius formula was 0.08, and the correlation coefficient (r) calculated was 0.96. And for Bijker formula, the RMS error was 0.10 and the correlation coefficient was 0.97.

Using Van Rijn formula and the correlation coefficient (r) was 0.95 (Fig. 3). The RMS error between the measured and calculated LSTR was 0.53. The difference was due to the fact that even

though the currents vary with depth especially for the low-wave energy coasts (Kumar et al., 2002), the longshore currents were measured only at the surface and used in calculations.

Fig. 3 shows the predicted versus measured transport rates by the used formulations. The CERC equation has a tendency to overpredict the measured transport rates, and it produced a large RMS error and the lowest coefficient of correlation. The Kamphuis formula and Bijker formula produced somewhat better predictions than the other two formulas tested.



Figure 3 Variations of measured and calculated sediment transport rate

Total calculated LSTRs along the surf zone using Kamphius formula (that has given the best results) after introducing the coefficient of correlation, varied from -69360,96 to 22251,84 m^3 /year with an average gross transport rate of -47109,12 m^3 /year and the transport was predominantly towards the west Fig 4.



Figure 4 Direction and magnitude of the LSTR

6. Conclusion

Sediment transport in the Boumerdes coastal region is driven primarily by the action of waves and winds. In general, waves from the predominant from the northwest to the southeast and from northeast to the southwest. and the Dominant Northern winds combine to drive sediment in a generally Easterly direction through the study area.

A number of empirical and semi-empirical sediment transport formulae have been developed for use in coastal applications. The commonly used formula of Kamphius(1991) is appropriate for use in the Boumedes region as it considers the processes of entrainment, transportation and deposition under the action of waves and steady currents. The total LSTR varied from -69360.96 to 22251.84 m³/year with an average gross transport rate of -47109.12 m³/year and the transport was predominantly towards the west.

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Energy Efficiency Optimization of the Drainage Pumping Station Baciu 1

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Rezumat: Proiectarea stațiilor de pompare urmărește nu numai satisfacerea cerințelor tehnice ale beneficiarului, ci și obținerea unui consum energetic minim în timpul exploatării. Scăderea randamentului, precum și creșterea cheltuielilor de reparații ale utilajelor, datorită vechimii în funcționare, conduc la costuri ridicate pentru apa pompastă prin S.D. Baciu 1. Se impune, deci, modernizarea stației. Noua stație trebuie proiectată astfel încat pierderile energetice să fie minime. Specific acestei stații de pompare este faptul că funcționează într-un domeniu larg al înălțimii geodezice, datorită variației cotelor Dunării, deci randamentul de funcționare a pompelor alese trebuie să se mențină la valori ridicate pe tot domeniul de funcționare. Lucrarea justifică creșterea eficienței energetice obținută prin reabilitarea stației.

Abstract: The engineering design of pumping stations aims not only the meeting of consumer's technical requirements, but also the entire possible energy economy during operation. The age of the existing drainage pump station Baciu 1 has resulted in unreliable service, increased costs and low energy efficiency, therefore it must be modernized. The new drainage pumping station has to be designed so that the hydraulic energy losses to be minimal. Specific for this pump station is that it runs over a large field of geodetic head and consequently, the efficiency requirements must be accomplished over the entire field. The paper presents the technical solution adopted for the new pump station and the gain of energy efficiency, by theoretically analyzing the operation points.

Keywords: drainage pump station, energy efficiency,

1. Introduction

Energy economy is a target for all engineers implied in industrial and agricultural design activities. Water pumping stations are among important electrical energy consumers, thus the price of electric energy has an enhanced contribution to the cost of pumped water. Therefore, the operation efficiency increasing of a pump station results in a significant saving of electric energy and also in a lower cost of pumped water.

Baciu 1 is a drainage pump station which takes water from the drainage canal Baciu and discharges it into the Danube, through Prival Vederoasa. This station is part of the Rasova-Vederoasa drainage system, in Constantza, with an important role in land reclamation. The system has been put into service in 1981 and since then it has been evacuating the humidity excess from a land surface of 1053 ha. The drainage pump station is essential for operation of Rasova-Vederoasa drainage system and the existing irrigation pump station has been in service for over thirty years. The age of the existing irrigation pump station has resulted in unreliable service, increased costs and frequency of maintenance repairs, and low energy efficiency, therefore it has to be modernized.

The rehabilitation of the drainage pump station was proposed on the basis of a drainage research, that means the study of the agricultural drainage system and its effects to arrive at optimal system design. There were required the same water flow rate and pumping head as the old station.

2. Existing Drainage Pumping Station Baciu 1

The building of the drainage pumping station is located behind the protection dam along the Danube side, at an elevation of 9,60m. The operation

parameters for the pump station were established on the basis of hydrometeorological data registered between 1976-1979.

Pumps operate only when the water level in the Danube exceeds 8m. Otherwise, water is

gravitationally evacuated from the drainage canal, through two ducts of 1400mm in diameter, placed near the station.



Fig.1.Longitudinal section through the old pump station Baciu 1



Fig.2. Plan view of the old pump station Baciu 1

The installed power of the station was of 180kW. The pumps and the hydraulic equipment (gate valves, check valves) were hosted in the building. The pumps individually operated. Each pump had a suction duct and a discharge duct of 1000mm in diameter, as it may be noticed in fig. 1 and 2.

The old pump station was equipped with two axial flow pumps, with horizontal axis, AR 785. Their electrical motors had 90 kW and 750r.p.m. The movement was transmitted from the motor to the pump by a reduction gear that reduced the output turning speed down to 410r.p.m.

Neglecting the quality decline in time, the theoretical efficiency of a pump, at the operation point of H=6m si Q=0,6 m³/s was =50%. That was the technological level at that moment. In time, the efficiency decreased due to the friction between pump parts in relative movement or due to inaccurate parts replacement. We don't know the actual pump

efficiency, but it's obvious that is smaller than 50%. It is well known that it is best to avoid any pump that has an efficiency of 55% or less. 55% efficiency is the industry standard used to estimate the performance of a pump when the actual efficiency is unknown.

Furthermore, the old gate valves and check valves introduced important energy losses, according to their coefficients of local head loss of 0,3 and respectively 3,7.

All these losses result in important energy consumption, which during the year 2006 was of 71,078 Mwh.

3. Proposed Drainage Pumping Station Baciu 1

Aiming to reduce the energy consumption for the water drainage it was proposed the rehabilitation of the pump station Baciu1. The main rehabilitation measures are: the replacement of the pumping aggregates with more performing ones, the replacement of the ducts and valves with new ones and the introduction of equipment for real time monitoring.

The first task for the designers was to determine the range in which the geodetic head varies. The geodetic head for the old station was established according to the data registered for the Danube water level (above 8m). These data may be seen in the table1.

Table 1: Number of days in which Danube water level was above 8m

Yeasr				
Level[m				
]	1976	1977	1978	1979
8-8,5	68	214	127	130
8,5 -9	31	133	102	85
9-9,5	3	84	49	49
9,5-10	0	65	0	12
10-10,5	0	26	0	0

The geodetic head varies between 0,5m and 2,9m according to the values of Danube water level above 8m and the minimal value of suction water level, of 7,5m. The minimal value of water level in the suction basin is imposed by the existing pumps

that have suction ducts mounted on the bank slope canal edge.

The Danube water level considerably aroused during 2005 and 2006. These years there were recorded exceptional levels, greater than those reached in 1977, when devastating flood took place in our country. So, the maximal geodetic head must be increased.

The solution adopted for the new pump station implied the replacement of the pumping aggregates with submersible ones. Beside the fact that the submersible pumps are much more performing, the solution eliminates the suction ducts resulting in lower head losses. It was chosen an axial type of pump, mounted in tube. This pump allows the minimal level in the suction basin to decrease to 6,73m. According to the above discussion, we considered that the geodetic head may vary between 1,27 and 4,87m. Consequently, the operating point of the new station moves into a larger field of pumping head.

The new type of pump was carefully chosen. It is also an axial flow type pump, but with vertical axis. The characteristics of the proposed pump are:

- Volume flow rate: $Q_p = 0,636 \text{ m}^3/\text{s};$
- Pumping head: $H_p = 6 m$;
- Power: P = 58 kW;
- Turning speed: n=985 rpm.



Fig .3.Longitudinal section through the new pump station Baciu 1



Fig.4.Plan view of the new pump station Baciu 1

The value of the turning speed is greater. The specific speed of the new pump is $n_d=21,4$ instead of 8,7 for the old pump. It is known that an axial flow machine, whether a pump, turbine, or compressor, is more efficient at high specific speeds (high flow rate, low head).

These submersible pumps will be mounted in a new hydrotechnique construction, on the edge of the suction basin. The connection between pumps and installation inside the building is made of two metal ducts of 800mm in diameter. The check valve and the adjusting valve are mounted inside the builsing and they are both of buttefly type. The hydrodinamic shape of the buttefly obturator decrease the coefficient of local head loss at a value around 0,23. The discharge ducts continue outside the station building at the same diameter and 30,2m length, passing under the dam. The schema of the new station is represented in fig.3 and 4.

The new configuration of the station eliminates the priming installation, which is no longer useful. This is another source of electrical energy saving.

The main advantages offered by the new solution consist of :

- reduction of energy losses by the renewing of the hydraulic equipment (ducts, armatures, fittings);

-elimination of aspiration and of the priming pumps;

-possibility of pumping water at a lower level in the suction basin;

-automation and monitoring of the pumps operation.

-possibility of pumps conservation inside the building, when they are not in operation for a long time.

4. Energy saving. Results and Discussions

The most eloquent indicator of performance, from the energetic point of view, is the specific energy consumption coefficient, $C_e(kwh/1000m^3)$:

It allows the comparison between the new and the old installation.

Ce=2,725 H/
$$\eta$$
 (1)

where H-pumping head, [m]; -efficiency,[%].

It

indicates the energy consumed during an hour for lifting an unit of volume flow rate at the head H. It is calculated for 1000m³ of pumped water.

Considering the relation (1) and the theoretical values for the pumps efficiency, η , we determined the coefficient of specific energy at two operation points of the old pump station:

 $Ce=42 \ kwh/1000 \ m^3$ at the total head of 6m;

 $Ce=25 \ kwh/1000 \ m^3$ at the total head of 4,2m.

The actual average value, calculated for the year 2006 was : $Ce=46 \ kwh/1000 \ m^3$. It is obvious that the decrease in performance led to a greater energy consumption than estimated.

The electrical energy saving due to the modernization of the pumping station SD Baciu 1 can't be quantified without the analysis of operation points of the pumps. According to [1] both hydrodynamic efficiency and shaft efficiency of the pump rise with the Reynolds number. If the old pump operated at a Re= 4.10^5 , the hydrodynamic of the new pump leads to a number Re= 7.10^5 .

The two new pumps are identical and their installations are identical too. So, the hydraulic resistance modulus is the same for both lines $M=1,9m^{-5}$ s², calculated for a medium use of the ducts. The operation points were graphically determined, as it may be seen in fig.5. For each main possibility of pump operation (H_{gmin}, H_{gmed}, H_{gmax}), were determined the volume flow rate, the total head and the efficiency.



Fig.5. The determination of the main operation points for a pump

The coordinates and the efficiency for each point are:

$$F_{1}Q = 636l/s$$

$$H = 6m$$

$$\eta = 79\%$$

$$F_{2} \quad Q = 700 \, l/s$$

$$H = 4,2 m$$

$$\eta = 73\%$$

$$F_{3} \quad Q = 770 \, l/s$$

$$H = 2,5 m$$

$$\eta = 60\%$$
(2)

The pumps may function at any point between the limits F1 and F3. It may be noticed the increasing efficiency for the entire operation range.

The operation points F1 and F2, determined for the new pump, have respectively the same pumping head as the known operation points of the pump AR785, consequently we can compare the specific energy consumption coefficient, $C_{e..}$ Thus, the new coefficients are:

$Ce=21,6 \text{ kwh}/1000 \text{ m}^3$ at the total head of 6m;

 $Ce=17,5 \text{ kwh/1000 m}^3$ at the total head of 4,2m.

The maximal value for the coefficient C_e is less than half of the average energy consumption coefficient in 2006. A significant decrease of electric energy consumption was obtained, which means lower costs for the same amount of pumped water.

5. Conclusions

The decision to rehabilitate a pumping station is imposed by the low operation efficiency and frequently repairs which lead to high costs of the pumped water.

Considerable care needs to be taken by the designers for properly choose the most suitable type of pump and to minimize the energy losses on the pumping system.

The optimal solution to rehabilitate the drainage pump station Baciu 1 was adopted as a result of a detailed technical and financial analysis, comparing energetic parameters and the cost / pay– back for various options.

In order to accurately determine the magnitude of potential energy savings from the new pumps it was developed a rigorous study of the operation efficiency taking into account the wide variation of the geodetic head, which is specific to this pump station. The specific energy consumption coefficient proved to be the most suitable amount to compare the old and the new station operation. The lack of information about the characteristics of the old type of pumps limited the possibility of comparison. Therefore, there were investigated the operation points known for the old pumps, at 6m and at 4,2m pumping head. The specific energy consumption coefficient calculated for these operation points revealed a decrease at half the electric energy consumed before the replacement of the pumps and ducts.

The modernization of the pumping station involves three main types of cost savings:

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-electric energy savings from the pumping system (pumps and ducts);

-savings made by the elimination of repair needs;

-savings due to automation and monitoring of the pumps operation.

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STUDY REGARDING THE BEHAVIOUR IN EXPERIMENTAL TESTS ("ON STEND") UNTIL FAILURE FOR PRECAST REINFORCED AND PRESTRESSED CONCRETE POLES

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Rezumat: Lucrarea prezinta efectuarea unor experimentari prin incercare pe stend pana la rupere a unor stalpi prefabricati din beton armat si beton precomprimat utilizati pentru linii electrice aeriene (L.E.A.). Studiile de caz se refera la stalpi centrifugati de tip SCP 10001 din beton precomprimat si tip SC 15007 din beton armat, incercati pana la starea limita ultima (rupere) la incovoiere si respectiv torsiune. Pentru fiecare tip de element, sunt interpretate rezultatele experimentale si prezentate concluzii cu privire la modul specific de comportare la cele doua tipuri de solicitari (incovoiere si torsiune). In final, comparativ, sunt prezentate concluzii generale referitoare la specificul de proiectare, modul de comportare in exploatare si comportarea stalpilor centrifugati de din beton precomprimat si beton armat, incercati pe stend pana la starea limita ultima la incovoiere si torsiune.

Cuvinte cheie: stalp centrifugat din beton armat sau precomprimat, incovoiere, torsiune, rupere beton.

Abstract: This study presents the achievement of the experimental tests (on "stend") until failure for precast reinforced and prestressed concrete poles used in electrical transport lines (L.E.A.). The case study referring to centrifugal pole unit type SCP 10001 of prestressed concrete and type SC 15007 of reinforced concrete, conducted until ultimate limit state of failure in bending and torsion. For each type of element, experimental results and conclusions are presented referring to specific behaviour at both types application (bending and torsion). Finally, comparatively, are presented general conclusions referring to specific of the design, behaviour in service phase of the centrifugal poles of reinforced or prestressed concrete, conducted until ultimate limit state of failure in bending and torsion.

Keywords: Centrifugal reinforced or prestressed concrete poles, bending, torsion, tests up to failure.

1. Introduction

The experimental tests (on "stend") for precast reinforced and prestressed concrete poles was effectuate for behaviour attestation in different work phase conducted until ultimate limit state of failure for quality certification of the products and/or for project attestation through type experimental test.

In this study, the experimental tests where effectuate for centrifugal pole unit type SCP 10001 of prestressed concrete and type SC 15007 of reinforced concrete (at S.C. SOMACO S.A. Doaga, Vrancea county, in october 2006 and july 2007. ISSN-1584-5990

The coordinator of this study and experimental tests was by Prof. Dr. Augustin Popaescu.

2. Type of equipment

For this experimental tests was observed the stipulations of the SR 2970:2005 and SR EN 12843:2005 standards, with following explanations: - the experimental tests where effectuate for poles with minimum 28 days age, but maximum 40 days;

- there where used horizontal stends;

- the measurement precision of the loading forces and for arrow values measurement is minimum $\pm 3\%$. It was used a dinamometer type PIAB-Sweden (max. © 2000 Ovidius University Press 3000 daN), and for the measurements of the specific deformations was utilised fleximeter (3 pcs.) with 0.10 mm minimum division;

- at the torsion experimental tests was used one hydraulics pressing machine type Hydramold (maximum 200 mm race, maximum force 16 tf);

- starting, opening and closing of the cracks was observed with fissures magnifying glass (with 0.01 mm gradations);

3. Case study. Centrifugal pole unit type SCP 10001 of prestressed concrete

3.1 Identification dates

• This case study presents analyses of the behaviour mode at bending and torsion experimental tests of centrifugal pole unit type SCP 10001 of prestressed concrete:

- Producer of the both poles: S.C. SOMACO S.A., Doaga, Vrancea county;

- Dates for experimental tests: october 2006.

• General technical characteristics of the pole:

- Reinforcement: 6TBP9 (7Φ3), concrete class: C 45/55;

- Length: 10.00 m, weight of the pole: 600 kg;

- The resistance limit state values (S.L.R.):

a. Bending: $M^{SLR} = 2822 \text{ daNm}$

b. Torsion: $M_t^{SLR} = 300 \text{ daNm}$

- The exploitation values:

a. Bending:

 $P^{E} = 263 \text{ daN}, M^{E} = 2170 \text{ daNm}, f^{cE} = 206.50 \text{ mm}$ b. Torsion:

 $P_t^{E} = 370 \text{ daN}, M_t^{E} = 185 \text{ daNm}$

- *The control values* for bending/torsion moment and control forces which are producing these values of the moments at experimental tests (on "stend") until failure phase:

a. Bending:

 $P^{C} = 478 \text{ daN}, M^{C} = 3950 \text{ daNm}$ b. Torsion: $P_{t}^{C} = 720 \text{ daN}, M_{t}^{C} = 360 \text{ daNm}$

3.2 Bending experimental test method [1], [2]

First, behaviour in service phase

- Until limit state from service phase (loading value: $P^E = 263 \text{ daN}$), was applied two loading stages: $P_1=100 \text{ daN} (M_1=0.38M^E)$, $P_2=200 \text{ daN}$

 $(M_2=0.76M^E)$, for each loading stage, loading was conservation constant minimum 5 min.;

- First loading cycle was applied until limit state from service phase $P_3=P^E = 263$ daN, loading was conservation constant aprox. 15 min. ($f_{exp.}=179.56$ mm, cracks was not opening), v. Figure 1;

- After unloading ($P_4=0$), remanence $f_{exp.}=10.30$ mm;

Second, ultimate limit state in failure

- Was restart loading cycle: loading stages $P_5=100$ daN ($M_5=0.38M^E$), $P_6=200$ daN ($M_6=0.76M^E$), $P_7=P^E=263$ daN ($M_7=M^E$); $P_8=300$ daN ($M_8=1.14M^E$), $P_9=350$ daN ($M_9=1.33M^E$), $P_{10}=400$ daN ($M_{10}=1.52M^E$), $P_{11}=480$ daN ($M_{11}=1.83M^E$), $P_{12}=525$ daN ($M_{12}=1.99M^E$), $P_{13}=550$ daN ($M_{13}=2.09M^E$), $P_{16}=560$ daN ($M_{14}=2.09M_F^E$);

- At limit state from service phase $P_7=P^E= 263$ daN (second loading cycle), was follow values: maximum opening cracks, $\dot{\alpha}_{f max}=0.10$ mm, $f_{exp}=194.20$ mm (Figure 1);

- The last loading stage forward failure phase, was $P_{13}=550 \text{ daN} (M_{13}=2.09 \text{ M}^{\text{E}})$. was $f_{exp.}=942.00 \text{ mm}$ and maximum opening cracks, $\dot{\alpha}_{fmax.}=0.60 \text{ mm}$;

- The failure of the concrete was realized at loading stage $P^u=P_{14}=560$ daN (superior value comparatively with control value, $P^C = 478$ daN), v. Photo 2;

Analyses of the results (v. Tabel 1 values)

• Behaviour in service phase:

- The maximum values of cracks opening :

 $\alpha_{f max} = 0.10 mm \le 0.20 mm.$

- The arrow value at service loading ($P^E=263$ daN) was $f^E=179.56$ mm, respective $f^E=0.87 \times f_c$ ($f^c=206.50$ mm is arrow control value at service phase).

• Ultimate limit state in failure:

- The experimental value which characterize the behaviour of the pole in bending, are:

 $M_u^{experim.}$ / $M^{SLR} = 4620/2822=1.64>1.5$: failure through arrow condition

M_u^{experim.} / M^C=4620/3950=1.17

- Failure through arrow condition:

 $f^u > l/35$ (1/35=236 mm and $f^u > 942$ mm)

 $f^{u}/l = 1/9 > 1/35$ (failure criterion)

• Observations:

- Bending experimental test method of centrifugal pole unit type SCP 10001 of prestressed concrete distinguished a commensurate behaviour as obtained values comparative with control values (minimal requirement for ultimate limit state in bending);

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- Ultimate limit state in failure in bending was considered at overtaking limit arrow value (1/35).



Photo 1, Photo 2. Centrifugal pole unit type SCP 10001 of prestressed concrete, in bending. Assembly organisation. Ultimate limit state of failure. Detail



Photo 3, Photo 4. Centrifugal pole unit type SCP 10001 of prestressed concrete, in torsion. Assembly organisation. Ultimate limit state of failure. Detail



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Figure 1. Centrifugal pole unit type SCP 10001 of prestressed concrete, in bending. Loading cycles P, M – experimental arrow (f)

Table 1. Ce	entrifugal pole	unit type SCP	10001	of prestressed	concrete,	in bending.
		Centralization	n of the	e results		

LOAD VALUE	VALUE	FLEXIMETER	F _i (mm)	EXPERIMENTAL ARROW VALUE	BENDING MOMENT	EVOLUTION OF THE CRACKS OPENING
P (daN)	F1 (mm)	F2 (mm)	F3 (mm)	f _{exp} . (mm)	M (daNm)	$\dot{\alpha}_{f}(mm)$
0	0	0	0	0	0	
100	52.40	0.10	0.10	51.07	825	
200	116.50	0.30	0.20	113.17	1650	
$(I) P^{E} = 263$	184.90	0.50	0.30	f ^E =179.56	M ^E =2170	
0	12.30	0.20	0.10	10.30	0	
100	67.60	0.30	0.10	64.94	825	
200	133.70	0.40	0.20	129.70	1650	
$(II) P^{E}=263$	200.20	0.60	0.30	f ^E =194.20	M ^E =2170	ά _f =0.10
300	235.20	0.60	0.30	229.19	2475	
350	333.80	3.50	0.40	307.79	2888	$\dot{\alpha}_{f} = 0.18 - 0.20$
400	453.40	3.50	0.50	426.72	3300	ά _f =0.25
480		3.50	0.50	695.00	3960	$\dot{\alpha}_{f}=0.40$
525		3.50	0.60	861.00	4332	
550		3.50	0.70	942.00	4538	ά _f =0.60
P ^u =560					$M^{u} = 4620$	bursting concrete

3.3 Torsion experimental test method [1], [2]

- Torsion moment was applied at 0.25 m from angle of the pole (v. Photo 3);

- Torsion moment was applied breeder value, start with a first value, $M_{1,t} = 80$ daNm, until ultimate limit state of failure (value: M_{4,t}=M_{r,t}=363 daNm), maximum value measured by experimental appliance;

- The result of the experimental test is torsion moment accordingly failure value, that is M_{r,t}=363 daNm (v. Photo 4);

- Destruction of the concrete of the pole was realized on 60 cm distance, from angle of the pole (v. Photo 4).

Analyses of the results

• Note:

- For evaluation torsion moment where considered only large concrete contribution, the results was a underevaluation for capable moment of the design;

- At torsion test method, SREN 12843 standard solicit only failure moment effective value, without to compare with control values.

• Ultimate limit state in failure:

- The experimental values which characterized pole behaviour in torsion, are:

$$M_{rt}^{experim.} / M_t^{SLR} = 363/300 = 1.21$$

 $M_{rt}^{experim.} / M_t^{C} = 363/360 = 1.01$

• Observations and conclusions:

- Insufficiency transverse reinforcement producing failure type cracky/fragile in torsion;

- In order to obtain a failure behaviour mode less fragile in bending, it is recommended a transverse reinforcement, in corelation with the variouse section on the lengh of the pole.

4. Case study. Centrifugal pole unit type SC 15007 of reinforced concrete

4.1 Identification dates

• This case study presents analyses of the behaviour mode at bending and torsion experimental tests of centrifugal pole unit type SC 15007 of reinforced concrete:

- Producer of the both poles: idem pole case type SCP 10001

• General technical characteristics of the pole:

- Reinforcement: 13Φ12 PC52, concrete class: C 40/50;

- Length: 14.00 m, weight of the pole: 1500 kg;

- The resistance limit state values (S.L.R.):

a. Bending: : $M^{SLR} = 6670 \text{ daNm}$ b. Twisting: $M_t^{SLR} = 264 \text{ daNm}$

tion values:

a. Bending:

 $P^{E} = 433 \text{ daN}, M^{E} = 5305 \text{ daNm}, f^{cE} = 224.00 \text{ mm}$ b. Twisting:

 $P_t^{E} = 406 \text{ daN}, M_t^{E} = 203 \text{ daNm}$

- The control values for bending/torsion moment and control forces which are producing these values of the moments at experimental tests (on "stend") until failure phase:

a. Bending:

 $P^{C} = 678 \text{ daN}, M^{C} = 8209 \text{ daNm}$ b. Twisting: $P_t^{C} = 740 \text{ daN}, M_t^{C} = 370 \text{ daNm}$

4.2 Bending experimental test method [1], [2]

First, behaviour in service phase

- Until limit state from service phase (loading value: $P^E = 433$ daN), was applied three loading stages: $P_1=100$ daN ($M_1=0.23M^E$), $P_2 = 200$ daN $(M_2=0.46M^E)$, P₃=300 daN $(M_3=0.69M^E)$, for each loading stage, loading was conservation constant minimum 5 min.;

- First cracks in concrete was the apparition at loading stage P₂=200 daN (maximum opening cracks, $\dot{\alpha}_{f}$ _{max.}=0.05 mm, f_{exp.}=67.65 mm);

- First loading cycle was applied until service phase $P_4=P^E = 433$ daN ($M_4=M^E$), loading was conserved constant aprox. 15 min. (f_{exp} = 301.97 mm, maximum opening cracks, $\dot{\alpha}_{f max} = 0.10-0.15$ mm), Photo 6;

- After unloading (P₅=0), remanence $f_{exp} = 17.18$ mm $(\dot{\alpha}_{f max} < 0.05 mm);$

Second, ultimate limit state in failure

- Was restart loading cycle: loading stages: P₆=200 daN $(M_2=0.46M^E)$, $P_7=440$ daN $(M_2=1.01M^E)$, $(M_2=1.38M^E),$ $P_8 = 600$ daN $P_{9} = 800$ daN $(M_2=1.84M^{E});$

- At limit state from service phase $P_7=P^E= 440$ daN (second loading cycle), where obtained the following values: maximum opening cracks, $\dot{\alpha}_{f max} = 0.20-0.30$ mm, $f_{exp} = 321.91$ mm;

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- The last loading stage forward failure phase, was $P_8=800$ daN ($M_2=1.84M^E$). $f_{exp}>1000$ mm and maximum opening cracks, $\dot{\alpha}_{fmax}=0.50\text{-}0.60$ mm; - The failure of the concrete was realized at loading stage $P_8>800$ daN (superior value comparatively with control value, $P^C = 678$ daN);

Analyses of the results

- Behaviour in service phase:
- The maximum values of cracks opening :



 $\alpha_{f max} = (0.10...0.15) mm \le 0.20 mm.$

- The arrow value at service loading (P^E =433 daN) was f^E =301.97 mm, respective f^E =1.35× f_c

 $(f^{c}=224.00 \text{ mm is arrow control value at service phase}).$



Starting and opening of the cracks in loading cycle (1): P=200 daN ($\alpha_{f max}$ = 0.05 mm) P=300 daN (α_{f} = 0.05-0.10 mm) P^E =440 daN (α_{f} = 0.10-0.15 mm)

Photo 5, Photo 6. Centrifugal pole unit type SC 15007 of reinforced concrete, in bending. Assembly organisation. Starting and opening of the cracks in loading cycle I (elastic phases).





Photo 7, Photo 8. Centrifugal pole unit type SC 15007 of reinforced concrete, in torsion. Assembly organisation. Ultimate limit state of failure. Detail



Figure 2. Centrifugal pole unit type SC 15007 of reinforced concrete, in bending. Loading cycles P, M – experimental arrow (f)

Table 2.	Centrifugal pole	unit type SC	15007	of reinforced	concrete,	in bendi	ng
		Centralizatio	on of th	e results			

LOAD VALUE	VALUE	FLEXIMETER	F _i (mm)	EXPERIMENTAL ARROW VALUE	BENDING MOMENT	EVOLUTION OF THE CRACKS OPENING
P (daN)	F1 (mm)	F2 (mm)	F3 (mm)	f _{exp} . (mm)	M (daNm)	$\dot{\alpha}_{f}(mm)$
0	0	0	0	0	0	
100	10.00	0.10	0.10	8.52	1225	
200	70.60	0.10	0.30	67.65	2450	$\dot{\alpha}_{f \max} = 0.05$
300	172.40	0.10	0.70	166.51	3675	$\dot{\alpha}_{f \max} = 0.05 - 0.10$
$(I) P^{E} = 433$	305.00	0.30	0.11	f ^E =301.97	M ^E =5305	$\dot{\alpha}_{f \max} = 0.10 - 0.15$
0	21.60	0.30	0.30	17.18	0	ά _{f max.} <0.05
200	135.00	0.30	0.60	128.37	2450	
$(II) P^{E} = 433$	325.00	0.30		f ^E =321.91	M ^E =5305	$\dot{\alpha}_{f max.} = 0.20 - 0.30$
600				575.00	7350	
P ^u =800				f ^u >1000	$M^{u} = 9800$	$\dot{\alpha}_{f max.} = 0.50 - 0.60$

• Ultimate limit state in failure:

- The experimental value which characterize behaviour of the pole in bending, are:

$$M_u^{\text{experim.}} / M^{\text{SLR}} = 9800/6670 = 1.47$$
, and

 $M_u^{\text{experim.}} / M^c = 9800/8209 = 1.20,$

- Failure through arrow condition:

 $f^u > l/35$ (1/35=350 mm and $f^u > 1000$ mm) $f^u / l = 1/12.5 > l/35$ (failure criterion)

• Observations:

- Bending experimental test method of centrifugal pole unit type SC 15007 of reinforced concrete distinguished a commensurate behaviour as obtained values comparative with control values (minimal requirement for ultimate limit state in bending);

- Ultimate limit state in failure at bending was considered at overtaking limit arrow value (1/35).

4.3 Torsion experimental test method [1], [2]

- Torsion moment was applied at 0.25 m from angle of the pole (v. Photo 7);

- Torsion moment was applied breeder value, start with a first value, $M_{1,t} = 186$ daNm, until failure value ($M_{4,t}=M_{r,t}=1186$ daNm), maximum value measured by experimental appliance;

- The result of the experimental test is torsion moment accordingly failure value, that is $M_{r,t}$ =1186 daNm (v. Photo 8);

Analyses of the results (v. Tabel 2 values)

• *Note*: idem pole case type SCP 10001

• Ultimate limit state in failure:

- The experimental values who characterized pole behaviour mode at torsion, are:

 $M_{rt}^{experim.} / M_t^{SLR} = 1186/264 = 4.49 \text{ and} M_{rt}^{experim.} / M_t^{C} = 1186/370=3.20$

• Observations:

- Insufficiency transverse reinforcement producing the cracky/fragile type failure in torsion;

- Low cover protection layer concrete for transverse reinforcement did not stroked the experimental behaviour, but may possible unfavourable influence the durability behaviour. In this case, is indicated to apply some measures for commensurate realisation of the cover protection layer concrete;

5. Conclusions

- The precast poles used in electrical transport lines (L.E.A.) are solicited prevalent in bending. A very important conditions is to limit the relative height of the concrete compressed zone ($\xi_{lim.}$), for non-cracky failure type;

- Bending test method of prestressed concrete column type SCP10001 and for reinforced concrete column type SC15007 distinguished a commensurate behaviour as obtained values comparative with control values (minimal requirement for ultimate limit state in bending application);

- For both type elements, the failure in bending was considerated at over limit arrow value;

- The absence or the insufficiency of the transverse reinforcement producing the cracky/fragile type failure in torsion;

- To assure a failure type less cracky/fragile in torsion, it is recommanded to include transverse reinforcement in correlation with various section on length of the pole;

- Low cover protection layer concrete for transverse reinforcement did not stroked the experimental behaviour, but could possible influence an unfavourable stricken ductility behaviour. In this case, it is recommended to apply some measures for commensurate realisation of the low cover protection layer concrete;

- In service phase, the precast poles of prestressed concrete has a favorable behaviour, without cracks at normal applications or working with closed cracks (assured a good durability of the reinforcement bars);

- The precast poles of reinforced concrete can crack starting with the transport and mounting phases, the cracks remaining open in service phase, with negative consequence regarding the reinforcement bars durability;

7. References

[1] SR 2970:2005, Precast columns of reinforced and prestressed concrete for overhead electrical lines. General technical requirements for quality.

[2] SR EN 12843:2005, *Precast concrete products. Mast and poles*

[3] SR EN 13369:2005, *Common rules for precast concrete products.*

[4] STAS 10107/0-90, *Civil and industrial buildings. Design and detailing of concrete, reinforced concrete and prestressed concrete structural members*

Contributions of the study reconsidering the behaviour at mechanical loads of sandwich panel made from polymeric compound with plated core materials

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Rezumat: Articolul studiaza proprietatile mecanice ale panoului sandvis si anume raspunsul in cazul incercarii la incovoiere. Dupa prezentarea generala a unui panou cu miez cutat se evalueaza experimental sageata maxima pentru un astfel de panou. Ulterior aceste rezultate se vor compara cu modelarea analitica și modelarea cu elemente finite. Toate trei metodele de concepere vor confirma posibilitatea utilizării poliesterilor armați cu fibre de sticlă la modernizarea elementelor de construcții. Se evidențiaza astfel posibilitatile de fabricare a unor produse performante din compozite cu matrice polimerica, prin utilizarea diverselor tehnologii.

Abstract: The article focuses on the properties of sandwich panel, precisely the reaction to bending. After a general review on plated core sandwich panel, the maximum co-flexure for such a panel is experimentally analyzed. Further on these results are compared with/to analytic modelling and finite elements modelling. The three conceiving methods will confirm the use of reinforced fibre glass polyesters to the refurbishing/modernizing of building elements. The manufacturing possibilities of some notable products made of composites with polymeric matrices, by using various technologies are also emphasized.

Keywords: sandwich panel, composite materials, polyester resin, polymeric matrix, fiber.

1. Generalization

Discontinuous sandwich panels with plated core, are structural elements with remarkable qualities from the point of view of the bearing capacity as well as the rigidity. Sandwich panels with discontinuous core, manufactured from polymeric compound materials, are used frequently used in fields like: aviation, constructions, naval construction etc. The main characteristics which determine their utilization in a variety of fields are the performances that these panels reach, like: high ratio between the mechanical properties and weight, good behaviour at corrosive actions, easy handling and manhandle. Also the geometric shape of the core and the panel in ensemble ensure them a convenient rigidity. The core of the sandwich panels can have diverse/various shapes: honeycomb with vertical or horizontal cells, plated or undulated.

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In the construction field these panels are used to make un-bearing curtain walls type or roof panels mostly under the flexure of exterior load actions. That is why it requires the utilization of a suitable procedure to evaluate the structural response to flexure.

The panel is made of composite materials reinforced with fibres. The sides of the panel and the core are stratified made from polymer reinforced with fibres on two directions.

2. <u>Experimental evaluation of the maximal</u> bent for a sandwich panel with a plated core

2.1. The process of making/bringing about the prototype panel.

The technological process used for making-up the polyester products reinforced with fibre glass is different in relation to the type of reinforcement material.

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The production flexibility is often the most important economical factor and if you fallow getting a high number of similar objects with the same mould the minimum total cost is obtained with automated methods which use casting under pressure, and if only some elements are necessary it is recommended the usage of methods with minimum moulds and equipment. The materials and the technological process must be correlated so that they reach the performance of the product at a reasonable manufacturing price using the quality of the material at maximum.

In the case of prototype products with relative big dimensions and complicated geometric shapes the contact forming process is recommended, also used in the experimental process presented further on, fig. 7.5.

Each layer of the panel is manufactured individually from polyester resin and fibre glass fabric. The volumetric fraction of the reinforcement is $V_f = 0, 5$.

The polyester resin reticulate after the reaction of co-polymerization with a free radical if it is initiated by a catalytic system that can release free radicals at room temperature. The catalytic system is made from methyl-ethyl-ketone peroxide and cobalt naphthenate. The hardening is done in a few stages: in the first stage the free radicals appear which react immediately with the inhibitors to ensure the resin a preservation period necessary to the impregnation of the reticulation accompanied by exothermia; after exceeding the exothermic peak the resin continues to harden for a few days.



Figure 1 – Sandwich panel with plated core experimentally made

When manufacturing the prototype Nestrapol 450, a polyester resin with high reactivity, good chemical and mechanical properties, was successfully used for cold formation by contact process.

The technological phases of making the individual layers were:

- arrangement of the first reinforcement layer on the plan (or plated) mould covered with deshuttering/striking foil;
- impregnation of the reinforcement layer with polyester resin (previously mixed with peroxide and naphthenate) by beating with a brush and rolling;
- repetition of the reinforcement layers until reaching the thickness of 4,5 mm for each subassembly of the panel (side or plated core);
- de-shuttering /striking the layers from the plan or plated mould;
- appliance of the polyester resin adhesive having identical composition with the composite material matrix;
- layer pressing and appliance of 200-250 N/m² weights for 24 hours;
- finishing the product and maintaining it in laboratory conditions until testing.

2.2. Testing the prototype panel at flexure from transversal evenly distributed loads

The mixture of the plated core panel and the nature of the composite material used trigger the following hypotheses: the exterior layers (sides) made of reinforced polyester with fibre glass are considered orthotropic in the xy plan; the exterior layers are thick and their local rigidity at flexure in relation with their own axes is negligible; because of the leaning conditions from the experimental program the plated core sandwich panel can be considered a wide beam leaned on his sides; shear rigidity of the plated core is very high in the XY plan and negligible in the ZY plan.

The main objectives of the experimental program are:

- a) the evaluation of the deformation characteristics of the panel under the action of the transversal loads, rigidity being one of the main designing criteria;
- b) the co-operation between the exterior layers and the plated core;
- c) the behaviour of the plated core, as a distance and stability element, under the action of transversal loads;
- d) the identification of the possible yielding modes of the sandwich panel, as part of an assembly or as components.

2.3. Adopted experimental conditions. Gadgets, hardware and experimental accessories

Panel (a), fig. 2, made conform the description in the precedent subchapter is leaned on cylindrical rolls (b) fixed on metallic profiles (c), connected to the metallic plates by solders (d). The diameter of the cylindrical rolls (32mm) satisfies the conditions imposed by standard regulation. Micro-comparators (e) with the measuring precision 0,01 mm, are fixed on the metallic supports (f). There were also used resistive electric transducers with a 20 mm measuring base made by INCERC Bucharest, a N2314 tensometric bridge and lead shot bags to achieve/obtain evenly distributed load (g).



Figure 2 – Organization of the prototype panel testing

2.4. Experimental process

The prototype panel was kept for 20 days in normal temperature and humidity conditions in the composite structure Laboratory of the Construction University of Iasi.

The transversal evenly distributed load was applied gradually, in stages considered significant for the evaluation of the panel behaviour in exploitation conditions, as closure element for walls and roofs.

The adaptation load value (550 N/m^2) was is equal to 50% of the stage corresponding to wind load, and the other load stages have been established according to the values used when designing the wall and roof panels.

The measuring points of the transversal movement and the specific deformation, figure 3, have been established at quarter and middle opening so that it describes almost completely the element's

Transversal	Transversal		Relative
load	movement		movement at the
q [N/m ²]	f [mm]		middle of the
	L/4	L/2	opening (f_{max}/L)
550	0,728	0,952	1/2100
1100	1,470	1,960	1/1020
1650	2,212	2,996	1/668

behaviour from transversal loads flexure.

Figure 3 – Prototype panel load sketch

2.4. Experimental process

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The measuring points of the transversal movement and the specific deformation, figure 3, have been established at quarter and middle opening so that it describes almost completely the element's behaviour from transversal loads flexure.

2.5. Experimental results

2.5.1. Transversal movement

The experimental values, table 1.1, of the transversal movement measured with microcomparators symmetrically disposed against longitudinal and transversal axes of the panel emphasizes the cylindrical flexure of the element under the evenly distributed load. The obtained values of movements in the symmetrical measuring points are near, which emphasizes/outlines a very good cooperation between the composite layers of the panels assembled by gluing with polyester resin.

The maximum relative co-flexure obtained at usual stages of short term loads are smaller than the ones recommended by usual design regulation (at roof panels it is accepted a relative co-flexure of 1/150 in areas without snow agglomeration and a 1/100 in areas with snow agglomeration).

Table 1.1 Transversal, absolute and relative movement

Co-flexure load curves from the figure 4 show that the panel has a quasi-elastic behaviour in the exploitation load period, and the calculus relations of mechanical movement of the structures can be used with acceptable precision.



Figure 4–Loading-unloading flexure curves from evenly distributed transversal charge

After the unloading the panel has returned to the initial geometric configuration, without notable remaining deformations.

The admissible limit of co-flexure at the middle of the opening was obtained at a charge of 7400 N/m^2 , almost 4,45 times larger than the load at which this kind of element is verified by calculus. The panel can take up exceptional loads due to snow agglomeration.

To evaluate the behaviour in time of the panel in conditions of excessive snow agglomeration the element was maintained under a constant load of 2600 N/m^2 for a period of 11 days. The daily movement record has lead to the laying out of the co-flexure growing curves in time, characterizing the gluey-elastic nature of the polymeric matrix material. The curves from figure 5, laid out with the measured movement values at the quarter and middle of the opening "describe" the behaviour of the panel at long term flexure under transversal charges.



Figure 5 – Transversal movement evolution under long term load

The curves from figure 5 emphasize a significant increasing movement in the first 6-7 days of loading, after this the tendency decreases. It is possible that under great transversal loads the movement stabilization period to be more extended.

The maximum transversal load of the panel was 24000 N/m², values at which flexures of 25 mm were measured ($f_{max}/L=1/80$) without the element to completely exhaust its bearing capacity.

2.5.2. Observed yielding shapes

To loads over 19000 N/m^2 local buckling shapes compressed at the superior layer on the cantilever section have been observed. This situation can be prevented through a suitable bracing system, eventually by punctual or continuous connections on the long side.

The adhesive connection by gluing with polyester resin has resisted in proper conditions to stress gliding between the plated core and the exterior layers to a load $q = 19000 \text{ N/m}^2$ which corresponds to a tangential tension of 0.82 N/mm^2 . At this value of the load local detachments have been observed especially in the support areas with big cutting forces.

Detachments became visible in loading interval $q = 19000...24000 \text{ N/m}^2$, the experiment being stopped at the last value.

It is assume that by changing the panel manufacturing technology or by using epoxies adhesive a better co-working between the component layers of the panel can be achieved.

The component strips of the plated core have not shown local buckling shapes until the appearance of the detachments caused by gliding between the layers; above 19500 N/m² value of the transversal load the local buckling shapes, specific to discontinuous plated core sandwich panel, became visible. Anyway these shapes of partial cession have been identified at loads much greater than the exploitation and calculus values, even in exceptional combinations.

2.5.3. Tension state in various load stages

The maximum normal tensions from the exterior layers of the panel have been determined through calculus and through resistive electric tensometry at 550 N/m^2 , 1100 N/m^2 and 1650 N/m^2 load stages.

Because of the panel constructive composition the extreme normal tensions are small, below the calculus resistance values of the polyester reinforced with fibre glass.

The signals of the literature confirm the design priority based on the rigidity condition (limit state of normal exploitation) in relation to the limit resistance state. Anyway instability shape coupling of thick exterior layer compressed with the panels general flexure require a more detailed study. The normal tension values experimentally determined and of those established through calculus are presented in table 1.2.

From the above table it can be observed that normal tensions established through calculus are very different from the experimental values between 5% -19%; the disagreement can be derived from the difference between elasticity modulus obtained on samples (and applied in theoretical relations) and the real elasticity modulus from the prototype panel. However the used manufacturing procedure (through contact) is known because of the unevenly properties of the resulting products.

Table 1.2 – Medium normal tensions

Normal and tangential tensions of the significant loading steps from experimental program are centralized in table 1.3.

Table 1.3 – Normal a	and	tangential	tensions	of the
loading main steps.				

Load q [N/m ²]	Significance loading stage	Normal maximal [N/mm ²]	Tangentia suppor [N/n At the joint between face and core	l stress in t zones nm ²] In the middle of the core
550	Adjustment	0,66	0,019	0,221
1100	Use (wall)	1,32	0,038	0,442
1650	Use (roof)	1,98	0,057	0,663
2600	Load	3,12	0,089	1,645
7400	Limit value of the deflection	8,88	0,256	2,960
19000	Starts the unfastening	22,8	0,657	7,650
24000	Maximal load	28,9	0,829	9,610

3. Conclusions

1. Experimental panel has been compared with analytic modelling and finite elements modelling. All three-conception models confirm that it is possible to use armed polyester with fibreglass when modernizing construction elements.

2. Proposed panel is a notable element with multifunctional characteristics (carrying capacity, rigidity, and thermal resistance) also displaying the possibility to place some installations in empty spaces of the folded core.

3. Calculus relations of the existing movement within the structure mechanics can be used

successfully if taken into account the mechanicgeometric characteristics of studied panel.

4. The used manufacturing technology is only the first stage/step in revealing the manufacture possibilities of some notable products made of composite and polymeric matrices. By changing the manufacture technology or by using epoxies adhesive you can count on a better co-working of the components and of the total thickness of the panel. Panels used for bigger openings and for different destinations can be build, especially in environments that require anticorrosive protection or areas that have magnetic and electric requirements.

5. The main designing/projecting criterion is derived through/by ensuring the rigidity condition because by ensuring the minimum thickness imposed by the technological manufacturing procedure the limit values of mechanical resistance can't be reached.

				Loa [N	ad q /m ²]			
		550		1100		1650		
	Measu	Norm	nal	Norm	al	Norm	Normal	
Area	ring	tensions		tensic	tensions		tensions	
	points	[N/m	m^2]	[N/m	m^2]	[N/m	m^2]	
		expe	theo	expe	theo	expe	theo	
		rim.	retic.	rim.	retic.	rim.	retic.	
	1,2,3	0,53	0,51	0,98	1,02	1,46	1,53	
Com	4,5,6	0,62	0,66	1,16	1,32	1,74	1,98	
pre ssed	7,8,9	0,48	0,51	0,95	1,02	1,39	1,53	
	1,2,3	0,57	0,51	1,09	1,02	1,55	1,53	
Stret	4,5,6	0,67	0,60	1,24	1,32	1,82	1,98	
ched	7.8.9	0.53	0.51	0.98	1.02	1.38	1.53	

^{6.} Improving the manufacturing technology, the thickness of the layers can be reduced and materials' resistance capacity can be used more efficiently.

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The behaviour of wood structures to fire. Comparison between the existent specialty literature (classic) until the apparition of Euro-code 5 and SR EN 1995 – 1 – 2 (Euro-code 5)

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Abstract: Articolul face o trecere in revista a proprietatilor materialului lemnos si plecand de la acest subiect se prezinta modul in care acestea sunt analizate atat de literatura de specialitate de pana la aparitia Eurocodului 5 cat si de Eurocod 5, cu accent pe proprietatile termice. De asemenea se evidentiaza o parte din noutatile pe care le aduce Eurocod 5 in studierea comportarii materialului lemnos la foc.

Abstract: The article is a review of wooden material properties and the way in which they are analysed within the literature before and after the appearance of Euro-code 5, focussing on the thermal properties. There are also outlined the innovations that Euro-code 5 brings when studying the behaviour/reactions of wooden material to fire.

Keywords: caloric power, thermal diffusivity, dilatation, carbonization, remanence section.

1. The structure of the wooden material

Wooden material has a wide usage in almost all branches of the economy as a finished product as well as a raw material (Paper and cellulose industry, textile industry – wool and artificial silk, musical instruments) or fuel.

By artificially modifying the properties of wood, new materials have been created, with an improved mechanical or fire endurance or regarding the behaviour to chemical agents and/or to atmospheric corrosion as well as to rottenness.

Wood has been given different definitions along time. A more ample definition is represented by the one that states that wood represents totality of secondary endurance, conduct and storage tissues, situated between the crust and marrow, which is the main part of branches, trunk and roots of wooden plants. It is a complex material composed of parts and elements distinct as shape, nature, sizes and properties.

Wood structure represents the totality of component parts (including the construction, morphologic nature and the proportion of

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components) including shape, nature, properties and their disposal inside wood.

From the point of view of chemical structure, wood is composed of organic substances like: 50% carbon, 44% oxygen and 6% hydrogen.

2. <u>The behaviour of wooden material at heat,</u> <u>within classic literature</u>

The burning phenomena represent an assembly of physical and chemical processes which follow one another in time.

The behaviour of wood to heat as well as to fire results from the study of its physical and mechanical properties.

Within classic literature the next concepts and indexes are defined:

2.1. <u>The caloric power of wood</u>, which is the physical characteristic that determines the behavior of wood to heat.

2.2. <u>Indexes that characterize the thermal properties</u> of wood:

- Specific heat, c;
- Thermal conductivity factor, λ ;
- Thermic diffusivity, a;

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- Thermal extension.

2.2.1. <u>Specific heat</u>, $c - of a body/item represents the amount of heat necessary to rise the temperature of one unit of its mass with <math>1^{0}$ C.

2.2.2. <u>Thermal conductivity factor</u>, λ of wood represents the amount of heat which passes through surface unit, in time unit, when the temperature decreases with 1^oC on the length or thickness of the body.

Thermal conductivity depends on humidity, temperature or direction of fibre, species/type and apparent specific weight of wood.

And for thermal conductivity there is a mathematical quantification according to the fibre direction.

2.2.3. <u>Thermic diffusivity</u> of wood, a, characterizes its capacity to uniform/standardize the temperature in its mass, at heat or cooling.

The higher the temperature the faster the uniformization is from one point to another.

Thermic diffusivity depends on the temperature and humidity of wood.

The most important factor is humidity.

Dry wood (Cells filled with air) is heated much faster than humid wood (cells filled with water). 2.2.4. Thermal extension

Another property of wood is thermal extension – mechanical property which is characterized through the linear extension factor which represents the variation of length unit at warming with 1° C.

This factor is smaller on longitudinal direction and greater on transversal direction.

3. <u>The behaviour of wood material at heat</u> <u>according to Euro-code 5</u>

Compared to classic literature, Euro-code encompasses the references given or not from specialized publications.

This European standard EN 1995 - 1 - 2 was elaborated by the Technical committee CEN/TC250 Structural Euro-codes, whose secretariate is detained by BSI.

Standard Euro-codes contain common structural design rules for a complete calculus of traditional or innovative structures and components. Because Euro-codes don't approach, especially, constructions with irregular shapes or special designing conditions, the designer must appeal to consulting some experts for supplementary considerations regarding these cases.

The used hypotheses are the general ones from the standard EN 1990: 2002 completed with the hypothesis that every system of passive protection against fires, considered in calculus, is properly maintained.

The general objectives of fire safety are to limit the risk regarding the individual property, associative adjacency and if necessary the direct exposed properties, in case of fire.

According to 89/106/EEC Norm regarding the construction products which establishes the main requirements for limiting the fire risk, such as: "Construction works must be designed in such a way that in case of fire:

 \checkmark The bearing capacity of the structure may be assumed for one determined period of time;

 \checkmark The out break or expansion of fire as well as smoke in the interior of the building are limited;

 \checkmark The inhabitants may leave the building or be saved on other ways;

 \checkmark The safety of rescue teams is taken in consideration".

According to the explicative Document "Siguranta in caz de incendii", the fundamental requirements may be observed by following the different stages of safety against fires available in the UE member states such as artificial fire scenarios (nominal fires) or natural fire scenarios (parametrical fires), encompassing both passive and/or active protection measures against fire.

That is the way the standard defines thermal conductivity, specific caloric capacity and soft essence wood density rapport.

The Euro-code 5 refers only to the necessary requirements for mechanical bearing, exploitation, durability and fire resistance in case of wood structures. It doesn't take into consideration other conditions like phonic and thermal insulations. It treats wood structures calculus in an accidental situation of fire exposure; it identifies the differences and completes the calculus under normal temperature conditions. It is also mentioned that only the passive fire protection methods are studied, in the structural design stage to maintain the bearing capacity of the structure in assembly and to limit the fire expansion in a more adequate manner. Also, Euro-code 5 takes into account the carbonisation effect defining the next notions:

 \checkmark The carbonisation line – the limit between the carbonised part and the remanent section;

 \checkmark Remanent section – the initial section of the element reduced by the carbonisation depth;

 \checkmark Actual section – the section of a resistance element, considered in the structural calculus at the fire action, based on reduced section method. It is obtained from the remanent section by eliminating the zero resistance and rigidity section parts;

 \checkmark Cession time of protection – the period of protection of an element against direct fire exposure;

 \checkmark Fire proof material – any material or combination of material applied to an structural element or to another element to improve its fire resistance;

✓ Normal temperature calculus – the last limit state calculus for the normal temperature according to EN 1995–1 –1;

✓ Protected elements – The structural elements which are measured to reduce the internal temperature growth and to prevent or reduce the carbonisation caused by fire;

✓ Carbonisation depth – represents the distance between the initial exterior surface of the element and the carbonisation limit and it is calculated according to the exposure time to fire and adequate burning speed.

The different requirements and performances, stated in Euro-code, can be specified either in terms of evaluating the nominal (Standard) resistance to fire action, established through national regulations regarding fires, or by referring to safety techniques in case of fire to establish the active or passive measures.

Additional requirements, for example:

 \checkmark Maintaining and installing fire protection equipments;

✓ Conditions regarding building occupancy or fire compartment;

 \checkmark Usage of approved insulating and protective materials, including their maintenance;

and they are not established by this document because they submit to authorizes authorities.

4. The properties of wooden materials

According to Euro-code 5, the properties of the material are mechanical and thermal. These are known as mechanical and/or physical-chemical properties in the classic literature.

4. 1. Carbonisation depth

Regarding the carbonisation depth this characteristic is more detailed in Euro-code 5. For all wood surfaces exposed to fire and, if necessary, for initially protected surfaces to fire exposure where the wood carbonisation appears after a period of fire exposure the effects of carbonisation must be taken into account. Carbonisation depth is also influenced by the properties of the material.

Further on Euro-code 5 states that sectional properties are calculated in relation to the real carbonisation depth including the rounding of corners.

The position of carbonisation limit is related to the 300 degrees isothermia - a valid hypothesis for most soft and hard essences.

It is considered that the burning speeds are different for:

 \checkmark The unprotected surfaces during fire exposure;

- The surfaces that were initial protected, before the protection cession;
- ✓ Surfaces that were initially protected but which are exposed after the protection cession

4.2. Unprotected surfaces during fire exposure

For standard exposure to fire, in case of unprotected surfaces, it is taken into consideration the carbonisation depth for the one-dimensional carbonisation written as $d_{char,o}$. This is compared with the theoretic carbonisation depth, $d_{char,n}$.

$d_{char,o} = \beta_{\circ} x t$

in which:

 $d_{char,o}$ = calculus value of carbonisation depth for one-dimensional carbonisation;

 β_{\circ} = one-dimensional burning speed for a carbonisation in standard fire exposure situation; t = fire exposure time.

$d_{char,n} = \beta_n x t$

in which:

d_{char,n} = theoretic carbonisation depth, that includes the corner rounding effect;

 β_n = theoretic burning speed whose amplitude includes the corner rounding and cracks effect.



Fig. 1. – One-dimensional carbonisation of a big section (one side fire exposure)



Fig. 2 – Carbonisation depth for one-dimensional carbonisation and theoretic carbonisation depth

Table 3.1 – Calculus values of burning speeds β_{θ} and β_n for wood, LVL wood panels and panels based on wood

	β_0 mm/min	β_n mm/min
a) Soft essences and beech Lamellate slabs with the characteristic density ≥ 290kg/m ³	0,65	0,7
Massive wood with characteristic density \geq 290kg/m ³	0,65	0,8

b) Hard essences		
Massive wood or lamellate	0,65	0,7
panels of hard essences with		
characteristic density of		
290kg/m ³		
Massive wood or lamellate		
essences with characteristic	0,50	0,55
density $\geq 450 \text{kg/m}^3$		
c)LVL	0,65	0,7
with characteristic density \geq		
480kg/m ³		
d)Panels		
Wood panels	0,9 ^a	-
Plywood	$1,0^{a}$	-
Wood based panels, other	0,9 ^a	-
than plywood		

^a The values are applied to characteristic density value of 450kg/m³ and a thickness of the panel of 20mm, to see 3.4.2(9) for other thickness and densities

5. Base requirements

The base requirements given in Euro-code 5 are also referring as in the literature appeared before the Euro-code to the carrying/bearing capacity of the element.

Therefore according to Euro-code, in case of imposed mechanical resistances during a fire situation, the structures must be calculated and executed so that they maintain the bearing capacity for a significant fire exposure period.

When fire divisions/sections are imposed, the closure elements of the building, including joints, must be calculated and executed so that they maintain the separation function for a significant fire exposure period. This requirement/desideratum is transposed in practice by ensuring that:

1.there is no integral cession;

2.there is no insulation cession;

3.thermal radiation of the unexposed fire side is limited.

If necessary, the structural deformation resistance is also taken into account (deformation limit state – a term also defined in the specialized literature) then; according to Euro-code the deformation criteria are also applied.

It is not necessary to consider the resistance structure deformation in the next situations:

 \checkmark The efficiency of the protective measures was tested according to the available standard procedures;

 \checkmark The separation elements are fulfilling the requests of a nominal fire exposure.

6. <u>A way/method of fire exposure</u>

The functions of structural elements are differentiated according to nominal fire exposure or parametric fire.

For standard exposure to fire conditions, the structural elements are obeying the next criteria:

 \checkmark Only the separation function: integrity even when insulation is imposed;

 \checkmark Only the bearing capacity function: mechanical resistance;

 \checkmark Both functions of separation and bearing capacity, specifying that the bearing capacity is considered fulfilled if it is maintained during the entire fire exposure period.

In case of parametric fire exposure, according to Euro-code 5, the problem is treated according to the temperature to which the element is exposed. That way, the bearing capacity function is maintained during the whole fire period including the diminution phase or the specified time period.

To verify the separation function, under the hypothesis that the normal temperature is 20° C the following criteria are applied:

✓ The medium growth/increase of the unexposed surface temperature of the construction is limited to 140K and the maximum growth/increase of the unexposed surface temperature doesn't exceed 180K in the warming period of the phase until the maximum temperature in the enclosure is achieved;

✓ The medium growth of the unexposed surface temperature of the construction is limited to $\Delta Q = 200$ K and $\Delta Q = 240$ K, the values finding themselves in the national annex, until the decline phase.

7. Joints

The section is applied to the joints between the elements under standard fire conditions and when this is not the case for fire resistances which do not exceed 60 min. The conditions are applied for joints realized with nails, bolts, cramps, screws, cogged rings, cut slabs and cogged slabs connectors.

It is also taken into account the symmetry, the action solicitation direction – axial or lateral and the manner of section embodiment – simple or composed.

Another analysis is referring to the type of the joints –protected or unprotected.

7.1. Unprotected joints

Fire resistances of unprotected joints with lateral wooden elements are present in the following table:

	Fire resistance				
	time t _{d,fi}	Regulations			
	min				
Nails	15	$d \ge 2,8 mm$			
Screws	15	$d \ge 3,5 \text{ mm}$			
Bolts	15	$t_1 \ge 45 mm$			
Cramps	20	$t_1 \ge 45 \text{ mm}$			
Connectors					
according	15	$t_1 \ge 45 \text{ mm}$			
to EN 912					
^a d is the diameter of the jointed element					
and t_1 is the thickness lateral element					

Euro-code 5 states that for clamps, nails or unprotected head screws joints, fire resistance times higher than the ones presented in table but not exceeding 30 min. can be calculated by growing/increasing the depth dimensions, lateral elements width and distances to the end and edging of joints elements, with this relation:

$a_{fi} = \beta_n k_{flux} (t_{req} - t_{d,fi}),$

Where:

 β_n – ignition speed, given in table;

 \mathbf{k}_{flux} – a coefficient related to the superior thermal transmission through joining element;

 \mathbf{t}_{req} – necessary time to fire resistance;

 $t_{d,fi}$ – fire resistance time of unprotected joint indicated in table who determines the time of/to fire resistance.

7.2. Protected joints

Protected joints can be with wooden panels, wooden base panels, or gypsum panels, for which the initial time of carbonisation is lower than the admissible time.


Figure 3 – Additional depths and distances against the end and the joint edging.

8. Conclusions

As a conclusion we can say that when dealing with wooden material Euro-code 5 brings new evidence concerning fire behaviour (as thermal and mechanical properties) simultaneously combining the technical specification for the construction products, as well as Romanian terminology with the European one.

A complex study concerning the behaviour of wooden material exposed to fire presupposes/implies a structural analysis concerning:

✓ breaking mode due to fire exposure;

 \checkmark the properties and stiffness of material /element closely related to temperature;

 \checkmark expansion and thermal deformation effects (indirect effects caused by fire).

From this view of point the standard Eurocodes contain structural designing common rules for traditional and international structures and components.

Europe Union member states recognize Eurocodes as reference documents which can be used as: 1. means to probe construction and engineering working conformity to the essential demands of Council 89/106/CEE Directive, especially fire resistance and stability demands (Essential demand No.1 – Mechanical stability and resistance and Essential demand No.2 – Safety measures in case of fire);

2. technical specification base (common language) to contract the working site and associated technical services;

3. technical specification background for each construction product.

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VRRulnerability and Risk – Evaluation Methods in Civil Engineering

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Rezumat: Analiza vulnerabilitatii este domeniul cel mai putin dezvoltat, fiind considerat la inceput de drum si unul din domeniile greu abordabile, necesitand cunostinte si resurse semnificative pentru obtinerea unor rezultate concrete. Analiza vulnerabilității in construcții, necesită eforturi în scopul elaborării de soluții viabile pentru cuantificarea riscului. Aceasta datorita faptului ca toate constructiile sunt vulnerabile la diferiti factori de risc, aspect ce necesita o abordare complexa pentru diminuarea riscurilor in scopul cresterii sigurantei constructiilor. Pentru a compara parametrii economici ai unei investitii si siguranta ei, este nevoie ca aceasta din urma sa fie corect cuantificata si sa fie exprimata in termeni economici. Lucrarea scoate in evidenta aspecte ale conceptului de vulnerabilitate legat de conceptual de risc si exprimat in termeni matematici si economici.

Abstract: The vulnerability analysis is the less developed field, being considered in it's early stage and one of the hardest broach, needing significant knowledge and resources for obtaining of concrete results. The vulnerability analysis of constructions requires great efforts to obtain viable solutions for quantifying the risk. This is due to the fact that all buildings are vulnerable to different types of risk factors, an aspect that requires a complex approach for risk reducing and to ensure the increasing of the constructions' safety. For comparing the economicity of an investment and its safety, is required that the last one must be correctly quantified and be expressed in economical terms. The work point out aspects of the general notion of vulnerability bound to the conceptual of risk and is expressed in mathematical and economical terms.

Keywords: risk evaluation, vulnerability, risk reduction, damage, social cost

1. General considerations

The intuitive understanding of the word safety appeared in the early ages. In Antiquity and in The Middle Ages, the safety of the structures was obtained exclusively by the constructor's experience, intuition and art, and after that the vulnerability appeared as a complementary necessity of the already used notion of safety.

The fundamental and scientifical elements of safety and vulnerability appeared only in the last 2 centuries.

The first definition of the safety concept, given as a fraction between the limit load and the effective loading (admissible) belongs to Rankine (1862). The significant values of the safety coefficient indicated by Rankine (2-10) [1] correspond to the low level of knowledge from that

period of time concerning the evaluation of loads and the assessment of the materials' resistance.

Nowadays, risk is "the possibility of having to endure damage (casualties), the exposure to a possible danger" (Romanian language dictionary) or "the possibility to suffer damage or loss, the exposure to a risk" (the American heritage Dictionary of the English language).

The idea of risk is probably world wide accepted, and the prove is the frequency of its usage. Moreover,

the risk is currently considered as an inseparable component of life.

Even if this situation is generally known, people's tendency is to consider risk as being the result of avoidable accidents and that this concerns "someone else". The accreditation of an objective random variability idea of the universe, the social experience accumulated and most of all statistically synthesized led to the need of reconsidering the strategies. A

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famous article (Wildavsky, 1979) had an important contribution to the understanding of the fact that not considering the risk is the biggest risk of all. The acceptance of a certain level of risk is inevitable, because the total elimination of risk is not possible, or in mathematical terms, it would imply infinite social spendings.

2. Vulnerability Measurement

The manner of considering the variability of the safety's main factors and the quantification of vulnerability shows the evolution of the vulnerability concept and of the safety coefficient, as well as the evolution of the calculation methods.

The methods that can be used [1] could be:

1. *deterministic methods* that include : the method of admissible resistances and the method of bearing capacity. These methods do not have a scientific foundation, and the values of the safety coefficient are established empirically by intuitive estimation, based on tradition and experience, partly covering the variability of the main factors on which vulnerability depends.

2. the *semiprobabilistic method* of the limit states is based on the analysis of the state in which a construction or an element of the construction loses its bearing capacity once it reaches the limit state.

3. *probabilistic methods* of calculation based on the probability theory of maintaining the capacity of a building in order to satisfy its function during the whole serviceability lifetime.

The main features of the limit state methods are:

- ➡ to systematically consider different possible limit states for a given construction/ building;
- ⇒ to consider in an independent way the variability of different factors which affect the construction's safety establishing, in consequence, the quantitative data which determine the safety level of the construction/building.

The limit state can be understood as a state that implies:

⇒ the reversible or irreversible loss of a construction's capacity to satisfy the

conditions of a successful exploitation for the intended destination, or

⇒ the appearance of human life threats or threats for material or cultural assets; whose state needs to be preserved based on the building.

The limit states are divided into 2 categories:

- ⇒ correspond to the bearing's capacity exhaustion or another irreversible loss of quality necessary during construction's exploitation.
- ⇒ *serviceability limit state* boundary states which correspond to the interruption of the capacity to assure a normal exploitation of the construction.

The main situations that can lead to the occurrence of limit states from the first category are:

- \Rightarrow different causes of failure;
- ⇒ loss capacity of a certain part of the construction or loss of the capacity of the whole construction;
- ⇒ loss of the position's stability (by sliding or by overturning);
- ⇒ states that make the construction inappropriate for using due to excessive permanent deformations or cracks.

The main situations that can lead to the occurancy of limit states from the second category are:

- \Rightarrow excessive static or dynamic displacements;
- \Rightarrow excessive cracks.

When analyzing these limit states it is necessary to consider all phenomena that lead to the appearance of the individual or combined states.

In computations, there are used different parameters (intensities, amplitudes, frequencies) for actions, resistances, modules of elasticity etc. Their values can be normal or design values. The normal values are reference values that are set by rules specified in Romanian standards and normatives. The design values are values used in different checks and take into account possible deviations in a detrimental way on the characteristic values.

The check of the construction's vulnerability in relation with the various limit states must be done taking in consideration:

⇒ a realistic hypothesis regarding the incremental level of the action's intensity until it reaches the necessary level of limit state for the given loading scheme; \Rightarrow the specific behavior of the structure in the considered stage.

The probability of damage of a structure (the structure's vulnerability) results by a comparison between the distribution of the actions and the distribution of the structural characteristics.

The scheme of this basic principle for checking the bearing capacity is illustrated in fig.



Actions Resistance ь

1a (Gauss' normal distribution hypothesis). One can notice that the maximum load is less than the minimum bearing capacity. Meanwhile, in fig 1b the minimum bearing capacity is smaller than the maximum stress so that the breaking, the yielding or the crashing of the structure occurs.

a - maximum stress < minimum bearing capacity;b – maximum stress > minimum bearing capacity; *1* – *minimum bearing capacity;* 2 – maximum stress; 3 – normal stress; 4 – normal bearing capacity; n – density of probability.

Fig. 1 The scheme of checking of the principle of bearing capacity

The overlapping zone of the 2 frequency functions for loads fs(x) and for resistances fr(x)indicates the probability of damage (accident, failure). By noting the coordinates of the crossing point of this 2 distributions with S and R, one can define the following probabilities:

the probability of bearing capacity loss, of ⇒ damage, accident or breaking (crashing or

collapse) P_r, contained between P(S>R) and $P(R \le S);$

the probability of safe behavior of a structure ⇒ against the random variation of the loading and resistance contained in the probability domain P(S>R) and P(R<-S) or the reliability L, which is a safety measure.



Fig. 2 Probabilistic model for R, S [2]

Because the probability domain distributed over the whole area limited by the distribution curve and the abscissa axle is equal with 1, then:

$$\int_{-\infty}^{+\infty} f(x) \, \mathrm{d}x = 1 \tag{1}$$

results that the probability of bearing capacity loss equals with 1 minus reliability:

$$P_r = 1 - L \tag{2}$$

meaning that considering the density of action's probability, results:

$$P(S > R) = 1 - P(S \le R), \tag{3}$$

and meaning that considering the density of resistance's probability, results:

$$P(R \le S) = 1 - P(R > S)$$
(4)

In a more suggestive way, one can obtain by using the condensed probability model R-S (resistance minus action effect) in which:

 \Rightarrow the damage probability represents the aria below the function f(R-S) from $-\infty$ to 0, defined by:

$$P_r = P(R - S \le 0); \tag{5}$$

⇒ the reliability represents the aria below the function f (R-S) from zero to +8 and which is defined by:

$$L = 1 - P_r = P(R - S > 0)$$
(6)



Fig. 3. Condensed probabilistic model for R, S

By using the fraction resistance-action effect R/S as a aleatory variable, one can obtain more significant expressions:

⇒ the probability of losing the bearing capacity:

$$P_r = P(R/S \le 1) \tag{7}$$

 \Rightarrow reliability :

$$L = 1 - P_r = P(R/S - 1)$$
(8)

For constructions of low importance, some reliability criteria used for different mechanical behaviors (cracking, rigidity, deformations etc) can be less severe than usual due to the fact that their damage does not endanger material or human loss.

In case of probability evaluation of stability and bearing capacity loss, one can consider the action's and

resistance's intensities as being random independent variables for a structure or its components.

The probability that a resistance element R can be subjected to a higher loading than S corresponds to the superposition zone $P(R \le S)$ and P(S > R) and can be expressed by:

$$[1 - F_s(x)]f_R(x)\mathrm{d}x\tag{9}$$

where: x = S, and:

$$F_s(x) = \int_{-\infty}^{x=S} f_s(x) dx$$
 (10)

$$F_R(x) = \int_{-\infty}^{x=R} f_R(x) dx$$
(11)

The probability of the action to be larger than a given value *S* can be expressed as follows:

$$P(x>S) = \int_{x+S}^{+\infty} f_s(d) dx = 1 - \int_{-\infty}^{x+S} f_s(x) dx = 1 - F_x(x)$$

In a probability case that the resistance can be equal with the given value x=R, it is: $F_s(x) dx$

Thus, the damage probability of a building can be expressed by:

$$P_r = P(R \le S) = \int_0^{+\infty} F_R(x) f_s(x) \, \mathrm{d}x$$
 (12)

This expression evaluates the area of the super positioned zone of the action's and resistance's probability density (bearing capacities) called convolution integral, the solution being obtained by graphical or analytical methods. In case of repeated application of the loading, the probability for the structure to resist is measured as the risk function of the bearing capacity loss h(n). The probability of a safe behavior (reliability) decreases when n increases.

In reality, the failure of an element can occur in different manners (bending, shear, torsion etc) considered independent and for a single random action S, acting singular or with a repeatability degree, n.

The damage probability can be expressed by:

$$P_r(l) = lP \tag{13}$$

$$P(l,n) = l.n.P_r \tag{14}$$

And the reliability:

$$L(l) = 1 - lP_r \tag{15}$$

$$L(l,n) = 1 - l . n. P_r$$
(16)

The safety coefficient has a number of various practical expressions based on probabilistic interpretation (Freudenthal, Cronell, Gauss).



Fig. 4 Definition of the safety coefficient

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3. Risk components

In the field of buildings safety, the risk is the potential detonator of damage, including loss of human lives.

The risk measurement is the product of: the probability of appearance of the consequences of failure and their corresponding damages. This is usually expressed by the risk rate (R), which expresses the possible annual damage. The defining relation is:

$$R = Pr \ge D \tag{17}$$

where: Pr – failure probability

D- the importance of the consequences produced by the failure.

The risk elements develop from the things presented above and consist of:

- ⇒ the probability of occurrence of an event that can cause damage and loss, which in buildings field is the failure probability;
- ⇒ the environmental conditions, respectively the level of loses and/or of the damage in case of appearance of the so called causeevent;
- ⇒ the measures to reduce the losses and/or damages after the event happened (the collapse of the construction).

We must mention that all the elements of risk vary in time, so, the problem is not how to proceed corectly at a given time, but to observe and to adjust all the risk-generating events.

The evaluation of risk is usually made separately on damage categories:

- \Rightarrow loss of human lives (L.H.L.);
- \Rightarrow recoverable material damage;
- ⇒ unrecoverable material damage (and sometimes invaluable).

The modern and evolved ways of expressing the accepted level of risk follows a model suggested almost 20 years ago (Oosthuizen, 1986) in a plan defined as ordinate by the probability of failure and in abscissa by the magnitude of the consequences. One point represents the risk of a certain construction, and the line represents a domain limit with the level of social acceptance.

4. The balance between risk and social cost

Decreasing the risk implies higher costs as the starting level is lower. The expenses of money and effort can be concentrated to increase the safety (studies, projects, changes regarding the structures, more expensive construction materials, more elaborate construction equipment and technologies etc), or in the direction of decreasing the level of damage (the protection and even the removal of some economical objectives, the interdiction of constructing in danger zones or in zones that can become in time, for one reason or another, possible danger areas, different manners of permanently supervising the construction site, alarming system etc).

To decrease the level of risk under certain situations implies a tremendous financial effort and taking it to the limit, a null risk implies infinite costs.

All these spending, whether the investor is a public institution or a private person, they all lead to social spending which have a direct influence upon the population (by taxes or by product and services costs). Finally, all these affect directly the living standards.

On the other hand, maintaining a high level risk also affects the population from the point of view of the possible consequences of the damage or by realizing the existence of the risk even if there is no harm done (physical problems, psychical discomfort) it is considered a serious social issue. This statement is based on the reaction of different communities around the globe when a various number of elements of risk appeared.

To solve this problem in a more adequate way it is the most important the population's decision, because they are the ones who suffer the most. The population's option that concerns the costs and the level of risk (so called "accepted risk") varies depending on age, culture level, sex, job, political and religious orientation and the time passed since the last major event occurred. So, the individual and collective opinions vary in time.

5. The settlement system of vulnerability and risk

Usually, the accepted level of risk (also implying the accepted safety) is not explicitly mentioned due to the fact that such a certain modality cannot be applied in many situations, or cannot be used and interpreted by many engineers who do not have a solid background regarding that particular branch. That is why the information regarding the vulnerability and risk assessment included in normatives and standards of a construction domain refer to the basic elements of engineering calculations, usually bringing out quite few data (even minimum). It is necessary for these technical settlements to be grouped in a general system of rules, prescriptions, standards, norms. All these must be complete and most of all, coherent. The

whole package of being complete refers to the coverage of all risk elements, therefore all fazes of realization and exploitation of a construction: studies, project elaboration, preliminary approval, execution and exploitation, (including quality system controls, alarming and evacuation) and all the domains of data entry (information, models and methods of calculation, safety coefficients, rules of execution, rules of exploitation and surveillance of the construction's behavior etc.).

It is recommended that all elements that will not be normed and will remain to the engineers' decision implicated in the construction project to be mentioned in an explicit way. The coherence demands that all the elements that contribute to the development of a construction to complete a system, not being allowed to superpose, the data, the methods or values among which there are conceptual contradictions, incongruence or without an authorized validation.

From an organizational – administrative and financial point of view, the manner of structuring and completing such a system of norms is the task of the public coordinator minister of that certain field of activity, and from a scientifically and technical point of view of the specialists of the highest technical level.

In the last years, in some countries and activity domains, the norms have also begun to explicitly mention normal failing probabilities or normal rates of risk. The higher the level of economical development is, the smaller the risk rates are, according to the maximum utility criteria.

By fully respecting the settlements and norms in construction's realization it must spare off of responsibility all the persons involved in the realization and exploitation of these constructions in case of incidents, accidents or damage that cannot be foreseen in a particular way, but whose occurrence, in average (including all the similar works from a long period of time) was socially accepted and stipulated by law.

6. Conclusions

The economical formula of low safety, which involves the existence of some risks that lead to a high vulnerability is represented by the expected rate of the damage expressed through the products sum of the probability advent of different types of possible dysfunctions (incidents, accidents, wrecks) and the damages value in case they appear.

Vulnerability points out how much people and their possessions are exposed to different types of hazards, indicates the level of damage that a certain phenomenon can produce and it is expressed on a scale between 0 and 1, 1 expressing total destruction of all goods and total loss of human lives in the affected area.

Risk was defined as being the probability of human exposure and its belongings to the action of a certain hazard at certain intensity. Risk represents the probable level of human lives lost, the number of injured people, the damaged produced to the owners and economical activities by a certain natural phenomenon or a group of phenomenon, in a certain place and at a certain time. The elements of impact consist in population, owners, lines of communication, economical activities etc, exposed to the risk in a certain perimeter.

The logical and natural consequence is that for every technical specialization field and for every beneficiary or product implementer the fundamental principle that defines the essence of engineering consists in the need to assure an eligible and rational equilibrium between the primary costs of the product (economy) and the exploitation costs like the ones derived from the damage caused by the nonfunctioning or/and the damage of this, is the most important part.

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Disasters and Risk Mitigation Measures

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Rezumat: Diminuarea dezastrelor este un domeniu ce cuprinde toate actiunile, toate masurile ce pot fi luate avand scopul de a reduce impactul unui dezastru inainte ca acesta sa se intample, inclusiv pregatirea si luarea masurilor de reducere a riscului pe termen lung. Include atat planificarea si implementarea masurilor de reducere a riscului, asociate cu hazarde naturale sau antropice, cat si procesul de planificare pentru masuri imediate ce pot fi luate in cazul aparitiei unui dezastru. Scopul general este acela de a sirul actiunilor de diminuare care pot fi considerate un raspuns la varietatea de hazarde naturale si antropice ce se pot ivi. Lucrarea prezinta diferentele dintre metodele active si cele pasive de reducere a riscului, precum si masurile de baza aplicabile in programele de planificare in vederea atenuarii dezastrelor.

Abstract: Disaster mitigation refers to all actions that have the purpose of reducing the impact of a disaster that can be taken prior to its occurrence, including preparedness and long-term risk reduction measures. It includes both the planning and implementation of measures to reduce the risks associated with known natural and humanmade hazards, and the process of planning for effective response to disasters which do occur. The general purpose is to outline the range of mitigation actions which can be considered as a response to the variety of natural and human-mad hazards which may be encountered. This article presents the difference between passive and active methods of risk reduction as well as the basic types of measures available for use in planning mitigation programs.

Keywords: disaster, hazard, vulnerability, risk reduction, threat, mitigation measures

1. General considerations

Mitigation means taking actions to reduce the effects of a hazard before it occurs. The term mitigation applies to a wide range of activities and protection measures that might be instigated, from the physical, like constructing stronger buildings, to the procedural, like standard techniques for incorporating hazard assessment in land-use planning.

The general aim is to achieve a significant reduction in the loss of life and material damage caused by disasters. Well trained organizations will play a central role in encouraging national governments and non-governmental agencies to tackle disaster related issues through projects focused directly on reducing the impacts of hazards and through incorporation of risk awareness as part of the normal operations of development projects.

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The great majority of casualties and disaster effects are suffered in developing countries. Development achievements can be wiped out by a major disaster and economic growth reversed. The promotion of disaster mitigation in the projects and planning activities of development protects development achievement and assists populations in protecting themselves against needless injury.

2. Reducing hazard vs. reducing vulnerability

Protection against threats can be achieved by removing the causes of the threat, (reducing the hazard) or by reducing the effects of the threat if it occurs (reducing the vulnerability of elements at risk).

For most types of natural disaster, it is impossible to prevent the actual geological or meteorological process from occurring: volcanoes erupt, earthquakes occur, cyclones and wind storms rage. The focus of mitigation policies against these hazards is primarily

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on reducing the vulnerability of elements that are likely to be affected.



Fig. 1 The concept of risk analysis

Some natural hazards can be reduced in certain circumstances. The construction of levees along the banks of certain rivers reduces the chance of them flooding the surrounding areas, for example, and it is possible to prevent known landslides and rockfalls from developing further by stabilizing land pressures, constructing retaining walls and improving drainage of slopes. The destructive agents of some natural hazards can be contained by engineering works or diverted away from important elements in channels and excavations. In some cases tree planting can be an effective way of either reducing the potential for floods and mudslides or to slow desertification. The potential for reducing the hazard level is given in each of the hazard profiles.

Obviously, preventing industrial accidents from occurring in the first place is the best method of mitigating future industrial disasters. Fire prevention, chemical spillage, technological and transportation accidents are all hazards that are essentially preventable.

In man-made risks of disaster the focus of disaster mitigation is in reducing or preventing the hazards from occurring. Engineering system safety is an important part of reducing risks form industrial hazards. A growing body of knowledge form the experience of long-established industries is applicable to the newly-industrializing regions.



Fig.2 Disaster risk as the product of hazard and vulnerability

3. Tools, powers and budgets

From the hazard profiles and the descriptions of actions that may be possible to reduce their effects, it is evident that protection is complex and needs to be built up through a range of activities undertaken at the same time. Protection cannot be simply provided by any single authority or agency. A government cannot provide housing that is windresistant for every citizen in cycloneprone areas. Governments can and do, however, influence individuals towards protecting themselves and the rest of the community. Governments can employ a wide range of tools and use their powers in many ways to influence the safety of the community. Legislative powers, administrative functions, spending and project initiation are all tools they can employ to bring about change. Powers of persuasion are sometimes classified into two types: Passive and Active. These are summarized below:

Passive mitigation measures

Authorities prevent undesired actions through controls and penalties by:

- \Rightarrow Requirement to conform with design codes.
- \Rightarrow Checking compliance of controls on-site.
- ⇒ Imposing court proceedings, fines, closure orders on offenders.
- \Rightarrow Control of land use.
- \Rightarrow Denial of utilities and infrastructure to areas where development is undesired.
- \Rightarrow Compulsory insurance.

Requirements of passive control systems

- ⇒ An existing and enforceable system of control.
- ⇒ Acceptance by the affected community of the objectives and the authority imposing the controls.
- ⇒ The economic capability of the affected community to comply with the regulations.

Active mitigation measures

Authorities promote desired actions through incentives like:

- ⇒ Planning control dispensations
- \Rightarrow Training and education
- ⇒ Economic assistance (grants and preferential loans)
- ⇒ Subsidies on safety equipment, safer building materials, etc.
- ➡ Provision of facilities: safer buildings, refuge points, storage
- Public information dissemination and awareness raising
- \Rightarrow Promotion of voluntary insurance
- \Rightarrow Creation of community organizations

Active Programs

- ⇒ Aim to create a self-perpetuating safety culture in areas of weak authority or poor ability to comply with existing controls.
- ➡ Require large budgets, skilled manpower and extensive administration.
- ⇒ Are useful in areas of low income, rural areas or elsewhere where there is no external jurisdiction over land use or building activity.

Safety standards, construction codes and building regulations form part of the normal apparatus that government use to help a community protect itself. One of the simplest measures for national authorities to take is to pass legislation for a national building code that requires new buildings and infrastructures to be resistant to the various hazard prevalent in that country. Some 40 earthquake-prone countries currently have seismic building codes for new construction. However, codes themselves are likely to have little effect unless the building designers are aware of them and understand them, and unless the community considers them necessary, and unless they are enforced by competent administrators.

The multiplicity of hazards and the different ways of reducing their various effects on the elements at risk is further compounded by the type of community powers and budgets available to the decision-makers. There is no standard solution to mitigating a disaster risk. For example, the construction of large scale engineering projects in Japan and other high-income countries to give protection against floods and volcanic debris flows, is not appropriate to mitigating similar hazards in developing countries. The enforcement of town planning regulations, and what is considered an acceptable level of interference by an authority on individual's right to build, varies considerably from one country to another, it varies from rural to urban situations and from one community and culture to the next.



Fig.3 Areas of action for disaster risk management

The prohibition of building houses on hazardous slopes may seem sensible but is unenforceable in cities where economic pressures to locate on such locations outstrip concerns of illegality. The right of a municipal engineer to inspect the seismic resistance of a building under construction may be accepted in major cities of a country but would be objected to in the more remote villages of the same province.

4. Community-based mitigation

It has been argued that governments and large development agencies tend to adopt a "top-down" approach to disaster mitigation planning whereby the intended beneficiaries are provided with solutions designed for them by planners rather than selected for themselves. Such "top-down" approaches tend to emphasize physical mitigation measures rather than social changes to build up the resources of the vulnerable groups. They rarely achieve their goals because they act on symptoms not causes, and fail to respond to the real needs and demands of the people. Ultimately they undermine the community's own ability to protect itself.

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An alternative approach is to develop mitigation policies in consultation with local community groups using techniques and actions which they can organize themselves and manage with limited outside technical assistance. Such community-based mitigation programs are considered more likely to result in actions which are a response to people's real needs, and to contribute to the development of the community, its consciousness of the hazards it faces and its ability to protect itself in the future, even though technically the means may be less effective than larger-scale mitigation programs. They will also tend to maximize the use of local resources, including labor, materials and organization.

Applying such community-based policies depends on several factors– the existence of active concerned local community groups and agencies able to provide technical assistance and support at an appropriate level, for example, are crucial to success.

Nevertheless, opportunities for community-based mitigation actions should always be sought in developing a comprehensive mitigation strategy. They will certainly be cheaper and may be more successful than alternative larger-scale programs.



Fig. 4 Disaster risk management as part of disaster management

5. Engineering and construction measures

Engineering measures are of two types:

- ⇒ those that result in stronger individual structures that are more resistant to hazards, and
- ⇒ those that create structures whose function is primarily disaster protection-flood control structures, dikes, levees, infiltration dams, etc.

Actions of the first type are mainly actions on individual buildings and structures and are sometimes referred to as "hardening" facilities against hazard forces. Improving the design and construction of buildings, agricultural structures, infrastructure and other facilities can be achieved in a number of ways. Design standards, building codes and performance specifications are important for facilities designed by engineers. Engineering design against the various hazards may include design for vibration, lateral loads, load surcharges, wind loads, impact, combustibility, flood resistance and other safety factors. Building codes are the critical front line defense for achieving stronger engineered structures, including large private buildings, public sector buildings, infrastructure, transportation networks and industrial facilities.

Disaster-resistance based building codes are unlikely to result in stronger buildings unless the engineers who have to implement the code accept its importance and endorse its use, understand the code and the design criteria required of them and unless the code is fully enforced by authorities through checking and penalizing designs that do not comply. A code has to fit into an environment prepared to receive it. Part of the measures necessary to achieve the "engineering" mitigation measures may include increased levels of training for engineers and designers, explanatory manuals to interpret the code requirements and the establishment of an effective administration to check code compliance in practice: the recruitment of ten new municipal engineers to enforce an existing code may have more effect in increasing construction quality in a city than proposing higher standards in building codes.

A large number of the buildings likely to be affected in a disaster, and those most vulnerable to hazards are not designed by engineers and will be unaffected by safety standards established in the building codes. These are houses, workshops, storerooms and agricultural buildings built by the owners themselves or by craftsmen or building contractors to their own designs. In many countries these non-engineered buildings make up a large percentage of the total building stock. The "engineering" measures that are needed to improve the disaster-resistance of non-engineered structures involve the education of builders in practical construction techniques. The resistance of houses to cyclone winds is ultimately dependent on how well the roofing sheets are nailed down, and the quality of the joints in the building frame and its attachment to the ground. Training techniques to teach builders the practicalities of disaster resistant construction are now well understood and form part of the menu of mitigation actions available to the disaster manager.

Persuading owners and communities to build safer, more disaster resistant structures and to pay the additional costs involved is required to make builder training effective. The building contractor may play a role in persuading the client to build to higher specifications, but unless this is carried out within a general public awareness of the disaster risk and acceptance of the need for protection, the contractor is unlikely to find many customers. Grant systems, preferential loans and supply of building materials have also been used as incentives to help improve the hazard resistance of nonengineered buildings. Legalizing land ownership and giving tenants protective rights also encourages people to upgrade building stock with security of tenure and a stake in their own future.

Apart form new buildings, the existing building stock also may need to be "hardened" against future hazard impacts. The vulnerability of existing buildings can be reduced to some degree by regular maintenance and the cost of adding strength to an existing building tends to be more expensive (and disruptive) than making new building design stronger, so strengthening is unlikely to be an economic option for the large majority of the building stock; for average buildings, with relatively short life expectancies(10 to 50 years), it may be better to take a long-term view of building stock upgrading, waiting until buildings come naturally to the end of their useful lives, demolishing them and buildings new structures in their place that conform to building code safety requirements.

For special structures, critical facilities or historic buildings with long expected life spans, retrofit strengthening techniques are now well established and a considerable amount of expertise has been developed in this field, though these are generally too costly to be useful in envelopment projects.

The engineering of large-scale flood control and water-supply measures is complex, lengthy and capital-intensive; and their construction frequently has adverse consequences for those they are intended to protect, for example some people may be forced off their land, land-use patterns may be changed and other adverse effects felt. Experience has shown that smallscale flood control measures which can be managed by community-based organizations can be effective in risk mitigation while simultaneously achieving other development goals. They tend to make use of local materials, labor and management resources to build on traditional mitigation knowledge rather than replacing it, and to enhance the community's own self-reliance rather than undermining it. Such measures can play an important role in disaster mitigation within integrated agricultural or rural

development projects.

6. Conclusions



For most of the risks associated with

natural hazards, there is little or no opportunity to reduce the hazard. In these cases the focus of mitigation policies must be on reducing the vulnerability of the elements and activities at risk. For technological and human-made hazards, reducing the hazard is, however, likely to be the most effective mitigation strategy.

Actions by planning or development authorities to reduce vulnerability can broadly be classified into two types-active and passive measures. Active measures are those in which the authorities promote desired actions by offering incentives-these are often associated with development programs in areas of low income. Passive measures are those in which the authorities prevent undesired actions by using controls and penalties-these actions are usually more appropriate for well-established local authorities in areas with higher incomes.

Community-based mitigation actions are likely to be responsive to people's real needs, to mobilize local resources and use local materials and contribute to the long-term development of the community, though in engineering terms they may be less effective than larger-scale capital-intensive alternatives.

Building disaster-protection takes time. It needs to be supported by a program of education, training and institution building to provide the professional knowledge and competence required.

Mitigation planning should aim to develop a "safety culture" in which all members of society are aware of the hazards they face, know how to protect themselves, and will support the protection efforts of others and of the community as a whole.

Diversification of the economy is an important way to reduce the risk. A strong economy is the best defense against disaster. Within a strong economy, governments can use economic incentives to encourage individuals or institutions to take disaster mitigation actions.

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Temperature changes actions on building structures. Concepts of Eurocode 1

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Rezumat: EN 1991-1-5 [1] indica principii si reguli pentru determinarea incarcarilor date de actiunile termice la cladiri, poduri si alte structuri, inclusiv elementele structurale. Standardul indica, de asemenea, principiile care se aplica in cazul inchiderilor de fatada si elementelor auxiliare (anexate) constructiilor. Lucrarea descrie variatiile de temperatura la nivelul elementelor structurale. Valorile caracteristice ale incarcarilor date de actiunile termice se utilizeaza la proiectarea structurilor supuse variatiilor zilnice si sezoniere de temperaturi. Actiunile termice nu trebuie luate in considerare la proiectarea structurilor care nu sunt supuse modificarilor climatice.

Abstract: EN 1991-1-5 [1] gives principles and rules for calculating thermal actions on buildings, bridges and other structures, including their structural elements. Principles needed for cladding and other appendages of buildings are also provided.

This work describes the changes in the temperature of structural elements. Characteristic values of thermal actions are presented for use in the design of structures which are exposed to daily and seasonal climatic changes. Thermal actions may not need to be considered in the design of the structures which are not exposed to climatic changes.

Keywords: thermal actions, shade air temperature, thermal resistance.

1. General considerations – design situations

Thermal actions shall be determined for each relevant design situation identified in accordance with [2].

Structures not exposed to daily and seasonal climatic and operational temperature changes may not need to be considered for thermal actions.

Structures in which thermal actions are mainly a function of their use (e.g. cooling towers, silos, tanks, warm and cold storage facilities, hot and cold services etc) are treated separately. Chimneys are treated in [3].

Thermal actions on buildings due to climatic and operational temperature changes shall be considered in the design of buildings where there is a possibility of the ultimate or serviceability limit states being exceeded due to thermal movement and/or stresses.

The elements of loadbearing structures shall be checked to ensure that thermal movement will ISSN-1584-5990 not cause overstressing of the structure, either by the provision of movement joints or by including the effects in the design.

Volume changes and/or stresses due to temperature changes may also be influenced by:

- \Rightarrow shading of adjacent buildings;
- ⇒ use of different materials with different thermal expansion coefficients and heat transfer;
- ⇒ use of different shapes of cross-section with different uniform temperature.

Moisture and other environmental factors may also affect the volume changes of elements.

2. Representation of thermal actions

Daily and seasonal changes in shade air temperature, solar radiation, reradiation, etc., will result in variations of the temperature distribution within individual elements of a structure.

The magnitude of the thermal effects will be dependent on local climatic conditions, together with

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the orientation of the structure, its overall mass, finishes (e.g. cladding in buildings), and in the case of building structures, heating and ventilation regimes and thermal insulation.

The temperature distribution within an individual structural element may be split into the following four essential constituent components, as illustrated in figure 1:

a) a uniform temperature component, ΔT_u ;

b) a linearly varying temperature difference component about the *z*-*z* axis, ΔT_{MY} ;

c) a linearly varying temperature difference component about the *y*-*y* axis, ΔT_{MZ} ;

d) a non-linear temperature difference component, ΔT_E . This results in a system of self-equilibrated stresses which produce no net load effect on the element.



Fig.1. Diagrammatic representation of constituent components of a temperature profile

The strains and therefore any resulting stresses are dependent on the geometry and boundary conditions of the element being considered and on the physical properties of the material used. When materials with different coefficients of linear expansion are used compositely the thermal effect should be taken into account.

For the purpose of deriving thermal effects, the coefficient of linear expansion for a material should be used. The coefficient of linear expansion for a selection of commonly used materials is given in table 1.

Material	$\alpha_T (\mathrm{x10^{-6}/^{\circ}C})$
Aluminium, aluminium alloy	24
Stainless steel	16
Structural steel, wrought or cast iron	12 (see Note 6)
Concrete except as under	10
Concrete, lightweight aggregate	7
Masonry	610 (see Notes)
Glass	(see Note 4)
Timber, along grain	5
Timber, across grain	3070 (see Notes)

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Note 1: For other materials special advice should be sought.

Note 2: The values given should be used for the derivation of thermal actions, unless other values can be verified by tests or more detailed studies. *Note 3*: Values for masonry will vary depending on the type of brickwork; values for timber across the grain can vary considerably according to the type of timber.

Note 4: For more detailed information see [4]. *Note 5*: For some materials such as masonry and timber other parameters (e.g. moisture content) also need to be considered. See Eurocode 5 and Eurocode 6.

Note 6 : For composite structures the coefficient of linear expansion of the steel component may be taken as equal to 10×10^{-6} °C to neglect restraining effects from different α_T values.

3. Determination of temperatures

Thermal actions on buildings due to climatic and operational temperature changes should be determined in accordance with the principles and rules provided in this Section taking into account national (regional) data and experience.

The climatic effects shall be determined by considering the variation of shade air temperature and solar radiation.

Operational effects (due to heating, technological or industrial processes) shall be considered in accordance with the particular project.

In accordance with the temperature components given in section 4, climatic and operational thermal actions on a structural element shall be specified using the following basic quantities:

1) A uniform temperature component ΔT_u given by the difference between the average temperature *T* of an element and its initial temperature T_0 .

2) A linearly varying temperature component given by the difference ΔT_M between the temperatures on the outer and inner surfaces of a cross section, or on the surfaces of individual layers.

3) A temperature difference ΔT_p of different parts of a structure given by the difference of average temperatures of these parts.

Note: Values of ΔT_M and ΔT_p may be provided for the particular project.

In addition to ΔT_u , ΔT_M and ΔT_p , local effects of thermal actions should be considered where relevant (e.g. at supports or fixings of structural and cladding account the location of the building and structural detailing. The uniform temperature component of a structural element ΔT_u is defined as:

$$\Delta T_u = T - T_0 \tag{3.2.1}$$

where T is an average temperature of a structural element due to climatic temperatures in winter or summer season and due to operational temperatures.

The quantities ΔT_u , ΔT_M , ΔT_p and T should be determined in accordance with the principles provided in 3.3 using regional data. When regional data are not available, the rules in 3.3 may be applied.

4. Determination of temperature profiles

The temperature T in expression (3.2.1) should be determined as the average temperature of a structural element in winter or summer using a temperature profile. In the case of a sandwich element T is the average temperature of a particular layer. Methods of the thermal transmission theory are indicated in section 4. When elements of one layer are considered and when the environmental conditions on both sides are similar, T may be approximately determined as the average of inner and outer environment temperature T_{in} and T_{out} .

The temperature of the inner environment, T_{in} , should be determined in accordance with Table 2 and the temperature of the outer environment, T_{out} , should be determined in accordance with Table 3 for parts located above ground level and Table 4 for underground parts.

Table 2. Indicative temperatures of inner environment

Season	Temperature T_{in}
Summer	T_1
Winter	T_2

Note: Values for T_1 and T_2 may be specified in the National Annex. When no data are available the values $T_1 = 20$ °C and $T_2 = 25$ °C are recommended.

The temperatures T_{out} for the summer season as indicated in Table 2 are dependent on the surface absorptivity and its orientation:

- the maximum is usually reached for surfaces facing the west, south-west or for horizontal surfaces,

- the minimum (in °C about half of the maximum) for surfaces facing the north.

Season	Significant factor		Temperature T _{out} [°C]
	Relative	0,5 bright light surface	$T_{\max} + T_3$
Summer	absorptivity depending on	0,7 light coloured surface	$T_{\max} + T_4$
	surface colour	0,9 dark surface	$T_{\max} + T_5$
Winter			T_{\min}

Table 3. Indicative temperatures T_{out} for buildings above the ground level

Note: Values of the maximum shade air temperature T_{max} , minimum shade air shade temperature T_{min} , and solar radiation effects T_3 , T_4 and T_5 may be specified in the National Annex.

If no data are available for regions between latitudes 45°N and 55°N the $T_3 = 0$ °C, $T_4 = 2$ °C and $T_5 = 4$ °C are recommended, for North-East facing elements $T_3 = 18$ °C, $T_4 = 30$ °C and $T_5 =$ 42°C for South-West or horizontal facing elements.

Table 4. Indicative temperatures	T_{out}
for underground parts of buildi	ngs

		U
Sezon	Adancimea sub nivelul terenului	Temperatura <i>T_{out}</i> [°C]
Summer	Less than 1 m	T_6
Summer	More than 1 m	T_7
Winter	Less than 1m	T_8
w inter	More than 1m	T_9

Note: Values T_6 , T_7 , T_8 si T_9 may be specified in the National Annex. If no data are available for regions between latitudes 45°N and 55°N the values $T_6 = 8^{\circ}$ C si $T_7 = 5^{\circ}$ C, $T_8 = -5^{\circ}$ C, $T_9 = -3^{\circ}$ C are recommended.

5. Temperature profiles in buildings and other construction works

Temperature profiles may be determined using the thermal transmission theory. In the case of a simple sandwich element (e.g. slab, wall, shell) under the assumption that local thermal bridges do not exist a temperature T(x) at a distance x from the inner surface of the cross section may be determined assuming steady thermal state as:

$$T(x) = T_{in} - \frac{R(x)}{R_{tot}} (T_{in} - T_{out})$$
(4.1)

where:

- T_{in} is the air temperature of the inner environment;
- T_{out} is the temperature of the outer environment;
- R_{tot} is the total thermal resistance of the element including resistance of both surfaces;
- R(x) is the thermal resistance at the inner surface and of the element from the inner surface up to the point x (see Fig. 4.1).

The resistance values R_{tot} , and R(x) [m²K/W] may be determined using the coefficient of heat transfer and coefficients of thermal conductivity given in [5] and [6]:

$$R_{tot} = R_{in} + \sum_{i} \frac{h_i}{\lambda_i} + R_{out}$$
(4.2)

where:

- R_{in} is the thermal resistance at the inner surface, [m²K/W];
- R_{out} is the thermal resistance at the outer surface, [m²K/W];
- λ_i is the thermal conductivity, [W/(mK)];
- h_i is the thickness of the layer *i*[m].

$$R(x) = R_{in} + \sum_{i} \frac{h_i}{\lambda_i}$$
(4.3)

where layers (or part of a layer) from the inner surface up to point x (see Fig. 4.1) are considered only.

Note: In buildings the thermal resistance $R_{in} = 0,10$ to 0,17 [m²K/W] (depending on the orientation of the heat flow), and $R_{out} = 0,04$ [m²K/W] (for all orientations). The thermal conductivity λ_i for concrete (of volume weight from 21 to 25 kN/m³) varies from $\lambda_i = 1,16$ to 1,71 [W/(mK)].



Figure 4.1: Thermal profile of a two-layer element.

6. Conclusion

The elements of loadbearing structures shall be checked to ensure that thermal movement will not cause overstressing of the structure, either by the provision of movement joints or by including the effects in the design.

Thermal actions on buildings due to climatic and operational temperature changes shall be considered in the design of buildings where there is a possibility of the ultimate or serviceability limit states being exceeded due to thermal movement and/or stresses.

Structures not exposed to daily and seasonal climatic and operational temperature changes may not need to be considered for thermal actions.

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Environment pollution, clean ports and costs estimation

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Abstract: In actual condition of development but also of economical regress; of technology evolution and also of its stagnation, the following question becomes legitimately : What is the role of environment management in the economical development of a port ? Intensification of activities specific to a port imposes both the estimation of ecological cost due to the environment pollution and the costs necessary to cancel their effects as a result of environment pollution . The work presents the problem of environment engineering solutions, ecological costs, the risk of pollution as well as necessary protection measures.

Key words: Strategical options, costs estimation, production costs, level of pollution, ecological costs.

Rezumat: În condițiile actuale de dezvoltare dar și de regres economic, de evoluție a tehnologiilor dar și de stagnare a acestora devine legitimă următoarea întrebare: Care este rolul managementului mediului înconjurător în dezvoltarea economică a unui port ? Intensificarea activităților portuare specifice impune deopotrivă estimarea costurilor ecologice datorate poluării mediului dar și a costurilor necesare eliminării efectelor rezultate în urma unei poluării a mediului. Lucrarea prezintă soluțiile inginerești pentru protecția mediului, costurile ecologice ,riscul de poluare și de asemenea măsurile necesare de protecția mediului.

Cuvinte cheie: Opțiuni strategice, estimarea costurilor, costuri de producție, nivelul poluării, costuri ecologice.

1. Environment and strategic developing options

The manager of a company or the commanding officer of a military units share the same ,,dissatisfaction" when they get an investments proposition for an ecological project, most of them being financially profitless, fig. 1.,[1].

	ECOLOGICAL	PUNISHED
	LEADER	GREY
HIGH	Very good image,	New equipment
COSTS	big investment	must be bought,
	policies	the image is very
	-	bad
LOW	CONFORMIST	UNINVOLVED
COSTS	Measures that does	He doesn'do
	not cost or are free.	anything

Fig.1 Strategical options. Which is the position you want to placed on?

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- The uninvolved is the person that does nothing, who doesn't spend a dime to environment protection and who has a grey or dirty image. The laws are changing in order to ensure a better environment protection. The one from this category will be placed in time in the punished category.
- The punished company does not succed to adapt to changes. It is punished by customers, which drops its services, demanding the existence of pollution control equipment. Substantial cost appear, but the company stays its grey color. The money was invested too late.
- The conformist is the one, which respect the law appealing to solutions that are not very costly to ensure, in the end, for a short period of time, a corresponding ecological image.
- The leader for a better image makes a lot of investments in order to obtain a superior efficient technology, which generates small © 2000 Ovidius University Press

quantities of waste materials. With time he can become the standard according to which the other company's value are determined.

It is obvious that a correct examination of costs, environment pollution and the protection costs to precisely avoid the pollution is not possible without taking care of risk idea. The costs caused by environment pollution can be classified using more criterions. A first difference appears between the pollution that generates remediable damages respectively irremediable damages. Another criteria makes the difference between the direct costs (tools and deteriorate products, production deficiency, etc.) and indirect costs.

The costs of pollution should also be taken into account (human and material losses) and the cost of correctional measures needed to reach the initial study (cleaning, etc.). Analyzing the diagram of strategical options it can be noticed taht close costs can oppose to long costs where is no action that can mend the respectively pollution. In order to offer a complete image about environment pollution this work will punctuate few aspects about this problem. Atmospheric pollution due to smoke, dust, gases, bad smelling substances. Cost estimation of the effect minimizing process depends on: air parameters (temperature, humidity, wind, etc.) and the amount of produced particles.

Water pollution is one of the main problems raised by ports in case of environment protection. Water pollution sources can be on the ground, can be ships or even the cargo handling. Soil pollution is recorded mainly in port and port areas. In a firs phase, the big majority of the activities led to polluting substances sprayed on the ground. The latent pollution provoked by the operational process has a risk that can be neglected either. Soil pollution depends in a big measure on soil nature. The hard or the clayey soils can limit the pollution extension. The sandy soils are characterized by a fast propagation both deeply and a the surface.

2. Cost estimation

Depending on the nature of the polluting phenomenon the costs can be accidental or structural.

The accidental costs correspond to ill-fated events whose cosequences on the environment pollution and the economical activities are serious. Such an event is characterized by: fires, explosions, important oil or another hazardous materials discharges. In these conditions cost estimation is quite difficult because of the high complexity of the variables that must be calculated. From this perspective, the ones in charge with naval transport activities, respectively port operating activities (maritime, fluvial, civilian or military) must take corresponding measures in the same time with the estimation of a corresponding budget in order to forestall these possible damages. If such an accident don't get enougt attention, the longterm results can be considerable and the risk enormous. These type of accident can temporarily stop the port operating activities causing very important economical and image prejudices. The structural cost coresponds to a constant dredging cost because of the current operation . We talk about events whose direct consequences are minor when they appear individually. On the contrary, when they appear are as dangerous as major occasional accidents. These structural cost came from the dust emanated by the bulk cargo terminals, from the smoke smitted by the industrial installation. The cost of environment pollution is a sum of accidental and structural costs for a determined period of time (by example a period of a year or the entire period when the protection measures are applied).

The risk

The risk of environment pollution, determined by the maritime or the fluvial transport activities, by port operating activities or by activities specific to the Navy impose for a correct evaluation of the environment effects an analyze of two elements: the frequency and the proportions. It is obvious that the risk level will increase with the frequency and with the proportion of the polluting phenomenon. The analyze of the risk situation must take into account at the previous experience in the operating domain of the company or the company with the same activities.

3. Environment protection measures

All the environment protection measures, no matter their nature (juridical measures, research programs, investment programs, etc.) have a cost of

their own. Considering these aspects, costs can be divided in two categories:

- a) direct costs, corresponding to structural investments:
- personnal instruction and recruitment;
- infrastructures constitution;
- equipment and installations maintenance and operation;
- the elaboration of juridical measures: laws, regulations, etc.
- b) indirect costs due to various aspects that caracterize the company activity or the analzyed structure:

- extra cost demanded by the environment protection regulations, commercial prejudices;
- delay in the port traffic or in the production production zones.

The problem of maritime environment pollution is characterized by specific aspects imposed by the international conventions so the costs due to the pollution are diverse.

Pollution due to transport activities, maritime and fluvial, requires the ships endowment with specilized equipment which will make possible onboard polluting sources annihilation, fig. 2



Fig. 2. Collecting storage and port transfer systems for the merchant and military 1,2,3,4 – polluting sources: oil polluted water effuent solid waste materials hazardous materials, etc.; CHT – Colecting storage and port transfer systems; OVD – Overboard averflowwing according to international convection.

Pollution due to port operating activities requires specialized ships and crafts, reception facilities for wasted materials from the ships according to MARPOL Convention. fig. 3.

In many cases, the port installation can process the wasted materials from the activities fulfilled on the ground (wasted materials storage stations, processing the plastic wasted materials, incinerators, etc.) We must say that these installations have retatively small acquisition costs but the maintenance costs are more important.

Pollution due to specific activities of the Navy is caracterized by major impact during military conflicts . During peacetime departments wits precise responsibilities in pollution field monitors the missions from this sector. By analyzing the aspects connected to atmospheric pollution, by example for the port areas, is drawn a conclusion according to which are needed tecniques to eliminate the small particles that appears when the bulk cargo is handled or stored (especially the coal). The atmospheric currents contribute to cargo dispersion.

Because of this, special measures should be taken like : sprinling, terminal delimitation with special panels ant the use of band conveyor. Obvious all these utilities claim new expenses for the environment protection.

From the previously described problems it result that the majority of environment protections measures have their cost.



Fig. 3 Waste materials management plan

A coherent and efficient policies, fundamented on real information claims that the investments and the running costs, including the production $costs(C_1)$ must be smallet than the total cost of environment pollution (C₂). To highlingt the different actions of various environment protection measures from the point of view of cost and efficiency, we will analyze the figure.



Fig. 4 Pollution level as a costs function

In this sense, it can be notice that to obtain a minim level of pollution (N_{pmin}) the costs are very big. Of course that every economical and social entity must define the acceptable level of pollution followed by the determination of reasonable values

of the polluting vectors (for atmosphere, soil and water).

The total cost line (C_T) records a minimal value for which we have an acceptable level of pollution . In this situation, the money needed to repair the damages (CMCE) as well as the total cost of pollution record the values C_{ce} and c_d . Economical speaking, if the activity developed presents a high risk for the environment taking measures against this risk will imply extra production costs.

In many situations the prevention and the protection activities is considered to have a smaller importance.

There is the possibility that such an option to situate us in one of the unfavorable situations presented on fig. 1.

4. Conclusions

The theoretical study and the experience from the environment protection field have demonstrated that in case of maritime or fluvial transport, of port operating activities or of specific activities to the navy the cost minimization is easier to realize since the very beginning. If a company reduces the pollution and produce less wasted materials the possibility of economy making is also reduced.

As can be noticed from figure number 5 the financial compensations are equilibrated on long term. It this point is reached the compary will focus on

93Florin Nicolae / Ovidius University Annals Series: Civil Engineering ,10 (2008)pollution causes rather then on pollution itself. Thepollution preventionrather like whe

costs are effective on long term when led to

pollution prevention rather like when its effects are reduced after pollution being produced.



Fig. 5 Ecological costs analize

From this graphic il can be notice that ecological activities must be situated above certain limit in order to be effective. Also the "S" shape of the curve offer smail profits at the beginning and big profits on long term with a decease in the next period. The problem previously decribed recommend in case of port operating units (commercial or military port) an analyze that include the costs and the environment pollution frequency but also the costs of its production (the analyze is destined for the main accidental or structural pollution of the environment from the field of activity).



Fig. 6 The analyze of environment protection measures

Depiste the fact that in many causes the evaluation are aproximate once the analyze is fundamented, it can costitute an important instrument to establish the priorities as a function of the impact and the cost of the measures to be tanken.

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Teoretical results for local elastic system protection against hydraulic shock

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Rezumat: Colectivul de autori ai prezentei lucrari si-au imaginat o solutie de protectie la soc hidraulic, in special la suprapresiuni, care este constituita dintr-un element elastic dispus longitudinal, (o camera de aer), concentric cu conducta, de lungime limitata, intercalat pe traseul conductei ce trebuie protejata.

In lucrare sunt prezentate cateva rezultate ale studiului realizat prin calcul numeric, referitor la eficienta unei asemenea solutii de protectie.

Abstract: The team authors of the present study has designed a solution of protection against the hydraulic shock, especially the overpressures, consisting of a longitudinal elastic element (an air room) which is concentrical with a length limited pipe, inserted on the pipeline that needs to be protected.

We compare some of the results of our numerical calculated study regarding the efficiency of such a solution of protection.

Keywords: water hammer, elastic system, unsteady regime, cavitation.

1. Introduction

It is known the fact that in the case of transitory movements the variations of pressure are directly proportional to the propagation speed of elastic waves (celerity). In steel pipes, in absence of some protection means, the celerity reaches values of 1000m/s. The presence of some small volumes of free air (about 2-3% of the volume of water) in the water that circulates trough the pipe makes the value of the celerity to diminish, reaching eaven to values of 100m/s. The diminution results from the fact that the water+air mixture is much more elastic

than the water without free air. In consequence the variations of pressure that result, will also be much more reduced. Starting from this, the authors team designed a disposition of prottection such as a room of air disposed concentric with the pipe and with a determineed length, the air being separated from the water by a rubber pellicle. It is expected that the celerity and also the variation of pressure to diminish considerable. In the present study the authores proposed to determine in an analytical manner, a way to calculate the celerity, take into consideration to the elastic dimensions and caracteristics of the room of air.



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2. Basics of Theoretical Calculus

To obtain a celerity formula, we start with expression in two ways of the variation of fluid mass in the time interval Δt , mass which is between two control sections, at distance $l = c\Delta t$, c - celerity, Δt - interval time.

$$m = \rho A \Delta l \Longrightarrow \Delta m = \Delta(\rho A) \Delta l \Delta t \tag{1}$$

If we consider the case of total instant closing of the valve, it will result:

$$\Delta m = \rho Q_0 \Delta t = \rho A v_0 \Delta t \tag{2}$$

From equalisations of the two expressions results first relation of celerity:

$$c = \frac{v_0}{\frac{\Delta\rho}{\rho} + \frac{\Delta A}{A}}$$
(3)

The proportion $\frac{\Delta\rho}{\rho}$, expresses the effect of water compression. It is express in the same way if there is not a protection device:

$$\frac{\Delta\rho}{\rho} = \frac{\Delta p \text{ final}}{E_W} \tag{4}$$

The proportion $\frac{\Delta V_w}{V_w}$ expresses the effect of air compression and it is find in this way:

$$\frac{\Delta V_w}{V_w} = \frac{\pi (D + \Delta D)^2 - \pi D^2}{\pi D^2} = \frac{\Delta D}{D} \left(2 + \frac{\Delta D}{D} \right)$$
(5)

 ΔD - diameter variation.

The last problem which has to be resolved is the determination of the relative variation of the diameter. We must consider that that the air compression and the flexibility of elastic element contribute to this phenomenon. We write the forces balance which actionates the water – elastic element – unit air:



Fig. 2. Section trough the disposition for protection before the hydraulic shock



Fig. 3. Section trough the disposition for protection after hydraulic shock

We note

$$\begin{split} L_{circle} &= \pi D; \quad \Delta L_{circle} = \pi \Delta D; \\ \varepsilon &= \frac{\Delta L_{circle}}{L_{circle}} = \frac{\Delta D}{D} = \frac{\sigma_c}{E_c}; \quad \Delta V_w = -\Delta V_{air} \end{split}$$

If we eliminate σ_c from the last second equations, it will be obtained:

$$\frac{\Delta D}{D} = \frac{\left(\Delta P_{final} - \Delta P_{air}\right)D}{2t_c E_c} \tag{6}$$

Because we don't know Δp_{air} , the last necessary relation refers at airs compression is considered politropic.

$$P_{air}V_{air}^{n} = (P_{air} + \Delta P_{air})(V_{air} + \Delta V_{air})^{n} =$$

$$= (P_{air} + \Delta P_{air})(V_{air} - \Delta V_{w})^{n}$$
(7)

Processing the prewious relation result:

$$2\frac{t_c E_c}{pD} \cdot \frac{\Delta D}{D} = \frac{\Delta p_{final}}{p} + 1 - \frac{1}{1 - \frac{n\frac{\Delta D}{D}\left(2 + \frac{\Delta D}{D}\right)}{\frac{4t_a}{D}\left(1 + \frac{t_a}{D}\right)} + \frac{n(n-1)}{2} \left[\frac{\frac{\Delta D}{D}\left(2 + \frac{\Delta D}{D}\right)}{\frac{4t_a}{D}\left(1 + \frac{t_a}{D}\right)}\right]^2}$$
(8)

We note

$$\alpha = \frac{t_c E_c}{Dp}, \ \beta = \frac{\Delta p_{final}}{p}, \ \delta = \frac{4t_a}{D} \left(1 + \frac{t_a}{D} \right), \ X = \frac{\Delta D}{D}.$$

We obtain

$$2\alpha X = \beta + 1 - \frac{1}{1 - n\frac{X(2 + X)}{\delta} + \frac{n(n - 1)}{2}\frac{X^2(2 + X)^2}{\delta^2}}$$
(9)

3. Results

The initial values are: $E_c=25x10^7 \text{ N/m}^2$; $E_w=2x10^9 \text{ N/m}^2$; $t_c=0.01 \text{ m}$; $t_a=0.25 \text{ m}$; $V_{air}=0.9816 \text{ m}^3$; $V_w=0.7853 \text{ m}^3$; D=1,00 m; $p_{air}=p_w=2x10^5 \text{ N/m}^2$; $\Delta p_{final}=4x10^5 \text{ N/m}^2$; n=1.3. Calculating the dimensionless parameters $\alpha = 12.5$; $\beta = 2$; $\delta = 1.25$; inserting the demensionless unknown $X = \frac{\Delta D}{D}$ and calculating the equation (9).



Fig.4. Diagram of T1, T2 by relative dilatation for different values of α

Tab.1.			
α	Х	Δα (%)	ΔΧ (%)
5.00	0 1 5 1		
2.00	0.101	50.00	34.82
7.50	0.112		
		33.33	27.27
10.00	0.088		
10.50	0.070	25.00	22.22
12.50	0.072	20.00	16.12
15.00 0.062	20.00	10.12	
15.00	0.002		

Getting an insight that the diammeter D and pressure p result from the instalation calculus in permanent state we studied the behaviour of protection with elastic element.

We give different values for the α parameter which means the efficiency variation when the term $E_c t_c$ varies. We obtain the conclusion that while the parameter α increases (respective teh term $E_c t_c$ increase), the elastic element deformation decreases.



Fig. 5. Diagram of T1, T2 by relative dilatation for different values of β

Suchlike we studied the elastic element behaviour by variation of β parameter. We cocluded that while the intensity of hydraulic shock increases, the elastic elements deformation inceseases. Because the pressure, results from permanent state calculus, the variation of β parameter represents the variation of intensity of hydraulic shock (Δp).

β	Х	Δβ (%)	ΔΧ (%)
1.00	0.0365		
1.00	0.0505	50.00	33.27
1.50	0.0554	20.00	55.27
1.00	0.000	33.33	24.55
2.00	0.0725		
		25.00	19.88
2.50	0.0905		
		20.00	16.20
3.00	0.108		



Fig. 6 . Diagram of T1, T2 by relative dilatation for different values of δ

1 ab. 3.			
δ	Х	Δδ (%)	ΔΧ (%)
0.96	0.0705		
		38.46	4.05
1.56	0.0740	3035	2.63
2.24	0.0760	5055	2:05
2.00	0.0750	25.33	1.29
3.00	0.0770	21.87	1 28
3.84	0.0777	21.07	1.20

Also we studied the behavior of elastic element by the third parameter noted $\delta = \frac{4t_a}{D} \left(1 + \frac{t_a}{D} \right).$ Because *D* results from calculus

of permanent regime results that the variation of δ parameter is realised by variation of parameter ta which represents the thickness of air cushion around the elastic element.

According to the tabel 3 we observe that by increasing the parameter δ , the relative deformation of elastic element ($\Delta D/D$) increases too.

4. Final Considerations

We analises the solutions to control of hydraulic shock using an longitudinal elastic element with finit lenght, inserted on the metalic pipeline (fig. 1).

In the initial situation the elastic element, which has the same section with the pipe, permits the flow of water as through the pipe. The air around the elastic element has a pressure equal with the pressure from the pipe. Thus this element doesn't introduce any suplimentary head loss when the instalation works in permanent regime.

Meanwhile the overpressure apear, due the hydraulic shock, the elastic element dilates until the dimensision of exterior diameter (D_{ext}) .

In this paper we considered the cocrete situation of instalation working. The results depend on 3 dimensionless parameters α , β , δ . The increasing of parameters β and δ leads to an increasing of deformation $\Delta D/D$. The increasing of parameter α leads to an decreasing of deformation

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Impact of the activities of Aker Tulcea Yards on groundwater

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Rezumat: În această lucrare sunt prezentate impactul pe care-l au asupra apelor subterane activitățile Șantierului Naval Aker din Tulcea. În urma analizelor calității apelor subterane se observă depășiri ale concentrațiilor față de limitele admisibile pentru: fier, crom, nichel, plumb și cloruri la forajul FGA2, amoniu la forajul, zinc la forajele FD10 și FGA2. Se observă că apa din forajul FGA2 este cea mai poluată.

Abstract: In this paper we present the impact of the activities of Aker Tulcea Yards on groundwater From the analises we observe great concentrations of: iron, chrome, nickel, lead and chlorine at drilling FGA2, ammonium at drilling FD7, zinc at drilling FD10 and FGA2. It is noted that water from drilling FGA2 is the most polluted.

Keywords: impact, groundwater, drilling, pollution.

1. Introduction

Aker Yards Tulcea is situated platform industrial city into a depression over its neighbours, therefore the quality of environmental factors in the area may be influenced major objectives neighbouring activity.

Appreciated the impact of Aker Yards Tulcea activities about the underground waters, were croped proofs from the existing drillings on emplacement.

Aker Yards Tulcea is on a platform industrial city into a depression over its neighbours, therefore the quality of environmental factors in the area may be influenced major objectives neighbouring activity.



Fig. 1. The map of the Aker Yard Tulcea

2. Description and results

To assess the impact of its activities Yards Aker Tulcea on groundwater samples were collected from drillings existing on-site. Data from the physical and chemical analyses were compared with the target values under STAS 4706 for surface water, category I and III, because national law does not provide rules on the quality of groundwater.

Table nr.1 – The quality indicators of groundwater

Nr. crt.	Determined indicator	Determine d values (mg/dmc)	Limit accorda STA	value in ance with S 4706	
	Drilling FD	10	Cat. I	Cat. III	
1.	Iron	0,768	0,3	1	
2.	Cadmium	0,707	0,1	0,8	
3.	Copper	0,1	0,1	3	
4.	Chrome	0,05	0,05	0,1	
5.	Zinc	0,217	0,01	0,1	
6.	Nickel	0,048	(),1	
7.	Cadmium	0,0005	0,005	0,2	
8.	Lead	0,004	0,05	0,1	
9.	Ammonium	2,83	1	10	
10.	Chlorine	42,54	250	400	
11.	Cyanide	0,0	0,01	0,2	
12.	CCO – Cr	29,2	10	30	
13.	Produced	0,00	,00 0,1		
	oilers				
	Dr	illing FD 7			
1.	Iron		9,64		
2.	Cadmium		0,554		
3.	Copper		0,031		
4.	Chrome		0,221		
5.	Zinc		0,03		
6.	Nickel		0,076		
7.	Cadmium		0,002		
8.	Lead	0,018			
9.	Ammonium	21,9			
10.	Chlorine	67,36			
11.	Cyanide	0,0			
12.	CCO – Cr	34,2			
13.	Produced		0,0		
	oilers				

Following the tests the quality of groundwater from the drilling FD 10 is found overshoot the limit value for water category III in the case of: zinc 0217 mg/dmc, compared with the limit value of 0.01 mg/dmc.

Nr. crt.	Determined indicator	Determined values (mg/dmc)			
	Drilling FGA 2				
1.	Iron	70,4			
2.	Cadmium	1,11			
3.	Copper	0,219			
4.	Chrome	0,768			
5.	Zinc	0,143			
6.	Nickel	0,468			
7.	Cadmium	0,014			
8.	Lead	0,29			
9.	Ammonium	16,1			
10.	Chlorine	290,7			
11.	Cyanide	0,0			
12.	CCO – Cr	155,9			
13.	Produced	0,00			
	oilers				
	Drilling FO	GC 4			
1.	Iron	1,15			
2.	Cadmium	0,13			
3.	Copper	0,0048			
4.	Chrome	0,143			
5.	Zinc	0,078			
6.	Nickel	0,055			
7.	Cadmium	0,0005			
8.	Lead	0,001			
9.	Ammonium	1,48			
10.	Chlorine	27,56			
11.	Cyanide	0,0			
12.	$C\overline{CO} - Cr$	14,2			
13.	Produced	0,06			
	oilers				



Fig. 2 Metallic ions concentrations at Metallic ions at drilling FD 10

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At drilling FGA 2 is found overshoot the limit values for water category III to:

■zinc: 0143 mg/dmc, compared with the limit value of 0.01 mg/dmc;

•iron: 70.4 mg/dmc over the limit of 0.1 mg / dmc;

 copper: 1.11 mg/dmc over the limit value of 0.8 mg/dmc;

nickel: 0468 mg/dmc over the limit value of 0.1 mg / dmc;

lead: 0.29 mg/dmc over the limit of 0.1 mg / dmc;

•ammonium: 16,1 mg/dmc over the limit of 10 mg/dmc;

•chemical oxygen demand 155,9 mg/dmc over the limit of 30 mg/dmc.



Fig. 3 Metallic ions concentrations at drilling FGA 2

The value of Fe from the graphic is three times higher than permissible limit.

At the drilling FD 7 finds overshoot the limit values for water category III to:

• iron: 9,64 mg/dmc over the limit of 1 mg/dmc;

chromium 0,221 mg/dmc over the limit of 0,1 mg/dmc;

ammonium: 21,9 mg/dmc over the limit of 10 mg/dmc;

• chemical oxygen consumption over the limit of 30 mg/dmc.

The amount of graphic for iron at drilling FD7 proved to be 10 times greater than the limit allowed.

At the drilling FGC 4 finds overshoot the limit values for water category III to:

 iron 1,15 mg/dmc over the limit value of 1 mg/dmc; • chrome 0,143 mg/dmc over the limit value of 0,1 mg/dmc.

The waters contain groundwater pollutants that may arise:

- of own activity: the zinc and ammonium (for which are some overshoot in sewage), the chlorine, chrome and other metals;

- the activity of other industry neighboring: the cooper, the iron (present in large cantity), but other metals too (chrome, nickel etc.).

The amount of graphic for iron at drilling FD7 proved to be 10 times greater than the limit allowed.



Fig. 4 Variations of ions concentrations at drilling FD 7

At the drilling FGC 4 finds overshoot the limit values for water category III to:

 iron 1,15 mg/dmc over the limit value of 1 mg/dmc;

• chrome 0,143 mg/dmc over the limit value of 0,1 mg/dmc.

The waters contain groundwater pollutants that may arise:

- of own activity: the zinc and ammonium (for which are some overshoot in sewage), the chlorine, chrome and other metals;

- the activity of other industry neighboring: the cooper, the iron (present in large cantity), but other metals too (chrome, nickel etc.).



Fig. 5 Variations of CCO-Cr concentrations


Fig. 6 Variations of ammonium concentrations



Fig. 7 Variations of chlorine concentrations

From the analises we observe great concentrations of: iron at drilling FGA2, chrome at drilling FGA2, nickel at drilling FGA2, lead at drilling FGA2, chlorine at drilling FGA2, lead at drilling FGA2, chlorine at drilling FD10 and FGA2. It is noted that water from drilling FGA2 is the most polluted. Water color is black, mainly due to chlor-iron and manganese and chromium sulphides and nickel.

3. Conclusions

Analysis of this case study entitles us to conclude:

1. The area in which it is situated on the site is a platform industrial city of Tulcea, on the right bank of the Danube. The land on which it is located, in large part, on an old swamp, over which were added various materials and a very thin layer of earth with reduced capacity to retain pollutants and to prevent water from infiltrating into groundwater. Further, unity is located at a much lower rate than the rest of the industrial platform, a position which facilitates migration of pollutants from the area by the construction site.

2. Neighborhoods construction site are, on the one side, drives on industrial platform, whose activities may influence the quality of environmental factors, including location Aker Tulcea, on the other side hand, is Somova of Reservation the Danube Delta Biosphere, an environmental special interest whose quality can be influenced by the work of construction site;

3. Profile of activity S.C. Aker Tulcea S.A. is the construction and repair of ships. Most activities are polluting the blast grit, metal covering and repair of ships.

4. The values determined for indicators of quality of groundwater were compared with the target values for STAS 4706 surface water (groundwater analysed arrive in the Danube), national legislation does not provide limits for this type of water.

5. At drilling FD10 has been made overshoot the limit value for category III in the case of zinc;

6. Concerning the drilling FGA2, here are the most overshoot limits water category III, namely:

zinc, iron, chrome, cooper, nickel, lead. ammonium and chemical oxygen consumption;

7. Concerning the drilling FD7 have been overrun the limit values for water category III in the case: iron, chrome, ammonium, chemical oxygen consumption;

8. At the drilling FGC4 have been overrun the limit values for water category III in the case: iron and chrome - 0143 mg/dmc.

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Analyse answered structurally from the seismic action

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Rezumat: Activitatea de proiectare curentă constă în determinarea forțelor seismice de nivel prin una din metodele acceptate de normativul P100/2006 și aplicarea acestor forțe static prin suprapunere cu greutatea proprie. Forțele seismice se aplică după cele două direcții în plan.

Nivelul forțelor seismice depinde de un coeficient de ductilitate care reprezintă capacitatea de disiparea a energiei.

Lucrarea își propune să determine pentru structurile în cadre, nivelul forțelor seismice pentru care structura trece din stadiul de structură multiplu static nedeterminată în mecanism prin formarea treptată a articulațiilor plastice.

Abstract: The design current activity is to determine seism forces by the methods of normative P100/2006 and the application of these static forces by overlap with it own weight. The seism forces are apply by the two plan direction. The level of the seism forces depends on a ductility coefficient that represent the capacity of energy dissipation.

The analyze proposes for the framed structures, the level of the seism forces, for that, the structure passes from the static indetermination structure, in mechanism in graduated plastically articulations.

Keywords: reinforced concrete, mesh, static equivalent forces, plastically

1. Introduction

In the first case the structure analyze by figure 1 was meshing in beam elements.

Weighted this structure with its own weight and with seism forces static equivalent will result the curved moments at each end of beam..

So that, the maximum bending moment for beams was 93.54 KNm, and for the beams having 25x40cm² section, we obtained an armed area of 9.42 cm².

The second analyze is about a meshing through the concrete was form by the plane finite elements with 8 nodes and also the frame by truss elements.

In case of quadrilateral elements it has been chose the solution with 8 nodes on each element, so that can be form the bending very well, because these elements are able to formulate the displacements by unlinear functions.

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There with, to be much accuracy in calculus, we used the 4 grade numeric integration.

For this case the load of beams is 44 KN/m.

Practically, previous study demonstrated that, even of a mesh with big elements, the uses of these elements for modeling the bending, involved on error of 20% maxim. The seism forces were apply like the ones equivalent static forces, distributed in the nodes of beams, and this forces have 7% of its own weight.

The reinforcement percents for columns are 1.1% and for beams 0.94%. The values of reinforcement areas were obtained from bending moments in the first study.

2. Numerical example

In the first step of calculus at these loads it obtained a maxim unitary effort of compression in concrete by 7.7 N/mm^2 (for columns). It can be compared this unitary effort by the bottom of the

central column with the unitary effort obtained in the same section from the analyze with beam

elements(the first case): $\sigma = \frac{N}{A} \pm \frac{M}{W}$ and it

obtains:

$$\sigma = \frac{28.4}{0.0045} + \frac{40}{0.09} = 6311 + 444 = 6755 KN / m^2 = 6.7N / mm^2$$



Fig.1 a) Mesh with beam b) Mesh with plane and truss elements.



Fig.2 a) The area where the stretch unitary efforts passed over maxim value admit in the first step calculus. b) remake of mesh

For the reinforcements the maximum unitary stress are 45.7 N/mm^2 .

The areas where appears the stretch maximum unitary efforts (for concrete) are presented in fig.2.

Maximum	Unitary stress for columns	Unitary stress for beam	Unitary stress for
displacements (m)	N/mm ²	N/mm ²	reinforcement
			N/mm ²
D _{ymax} =0.27E-2	σ(+)=4.0	σ(+)=5.3	σ(-)=45.7
D _{zmaz} =0.117E-2	σ(-)=7.7	σ(-)=5.4	

Table 1 (case 2) Displacements and unitary stress

It can observe that the frame has still a wide capacity reserve until the entrance flow, but the concrete touches 61% from maxim compression limit.

For modeling the concrete crack it remakes the mesh in the areas where appears the stretched efforts much more than 1.25 N/mm². We can see that in fig.2. This reshape can be done by taking off the stretched concrete area and keeping the frame.

Doing that it can observe the concrete crack for all area field beams, on the side of the stretched fiber and the crack concrete at the bottom of the columns (fig.3).

The maximum displacements up with an low percent and the maximum unitary stress for pillar lows from 7.7 N/mm² to 6.9N/mm² in the case of compression and the stretching from 4.0 N/mm² to 2.8 N/mm². This happened accordingly of redistribution effort state.



Fig.3 The areas where the unitary effort stretched are over the maxim admit value of the 2 step calculus. (the third case)

The following analyze is about the concrete elimination in the areas where the stretched maxim values are over 1.25 N/m^2 . These points are presented in fig.3.

The maximum values of displacements are presented bellow:

Maximum displacements	Maximum unitary stress	Maximum unitary stress for	Maximum unitary stress
(m)	for columns	beams	for frame
	N/mm ²	N/mm ²	N/mm ²
D _{ymax} =0.53E-2	σ(+)=6.5	σ(+)=8.0	σ (+)=42.9
D _{zmaz} =0.12E-2	σ(-)=11.2	σ(-)=-8.1	σ (+)=-68.8

Table 2 (case 3) Displacements and unitary stress

It can be observed that for the same level of seism static equivalent forces, accordingly of the efforts

redistribution, after the eliminate of the stretched

area, aren't the major modifications of the maxim

unitary effort at the compression. At the stretching

maxim unitary effort in the columns, the maxim values are under 1.8 N/mm².

The following analyze have a constant gradually of the level seism forces, reaching the maxim limit in concrete for an amplification factor of these forces equal with 2. For this level these forces equal with 2.

For this level of seismically static equivalent forces, the maxim tensions in the frame are at a minim value under 150 N/mm².

In the 4 th case the seism action isn't represented (introduced) like seism static equivalent forces, such as an accelerograme, practicaly used time integration. The seism exciting is introduced by an scaled accelerograme at 0.2g. Time step is 0.02s and the all accelerograme time is 15.98s.

Table 3 (case 4) Displacements and unitary stress

Because aren't major modifications of the unitary efforts distribution when we introduce in analyze the crack in concrete stretching areas, the analyses for time integration aren't considerated in this case. For a scaled accelerograme at 0.2g it obtained the following values for unitary stress and displacements.

Maximum	Maximum unitary	stress	Maximum	unitary	stress	Maximum	unitary	stress
displacements (m)	for columns		for beams			for frame		
	N/mm ²		N/mm ²			N/mm ²		
D _{ymax} =0.53E-2	σ(+)=6.5		σ(+)=8.0			σ (+)=42.9		
D _{zmaz} =0.12E-2	σ(-)=11.2		σ(-)=-8.1			σ (+)=-68.8	3	

If it is compared the analyze of the equivalent seism static forces with the time integration it can be observed the following differences:

-the maximum displacements increasing on horizontal from 0.31E-2 to 0.53E-2m

-the increasing unitary stress (at the bottom of columns) from 7.7 N/mm^2 to 11.2 N/mm^2

-the increasing maximum unitary stress in reinforcement from 49.5 N/mm^2 to 68.8 N/mm^2 .

The modifications are great both at the displacements and the maxim unitary stress in frame and in concrete, and we don't eliminate the concrete stretching area.

The seismic excitation applied in the 3 th case (seismic static equivalent forces) and the 4 th case, too, are accordingly to Constanta seismic area.

For the 4 th case it can be noticed the maxim values for the compression unitary stress by 11.2 N/mm², very closed by maxim permissible compression value of 12.5 N/mm².

For the beginning without doing another analyze, a simple calculus shows that it can be obtained the compression maxim limit in concrete for 0.22g. It's obviously clear that the concrete in the areas where has been reached the limit maxim compression, is

already cracked. We can say, that in these areas are formed plastically articulations. Initial, the analyzed structure has a degree static equal with $N = 3C - A - 2S = 3 \cdot 4 = 12$.

The following step increasing the seismic level, until 13 plastically articulations.

For the same scaled accelerograme at 0.25g studying the effort state, it observed reaching the

maxim value for $\sigma_{x1} = 12.1N / mm^2$ (compression) for the bottom of columns.

The great values are obtained for all the columns ends. And at the beams ends the reaching nearly at 10.0 N/mm² (σ_{x2}). These values are obtained without considerating the crack concrete in the stretching areas that accordingly to the linear analyze (the graduated increasing of the equivalent static seism forces) increases at least 12-20% of the compression tension. Because the maxim efforts first appear at the bottom of the pillars and then (after that) at the beams ends, we concluded that the structure failed by floor mechanism, due the plastically articulations at the columns bottom of the structure base. So the structure may failed only of 8 plastically articulations.

3. Conclusions

Studying by comparation the two analyze modalities: case 1,2,3 once and 4 case on the other hand, it can be observed that the tension state obtained by equivalent forces is much more less than the other of time integration. But if we take the time integration one, results that, yet the design of structure using the equivalent static seism forces, is possible to offer a bit reserve accordingly a needing resistance.

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Destination of the Stormwater using Spatially-Distributed Hydrological Model

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Rezumat: In acest articol este prezentata o aplicatie a modelului spaial distribuit TOPOG. Dupa o scurta prezentare a bazinului hidrografic pe care a fost testat modelul, este descris pe scurt modelul in sine, parametrii si rezultatele obtinute la calarea acestuia pe un set de date. Rezultatele obtinute confirma ca valorile scurgerii de suprafata sunt bine estimate.

Abstract: This paper presents an application of the TOPOG hydrological model, the model variables (parameters) and the results obtained through the calibration of the model. The model was tested for the Voinesti catchment that it is situated in western extremity of the Sub Carpathians of Curvature. The results obtained with this model confirm that surface runoff values are well approached.

Keywords: TOPOG, surface flow, simulation.

1. Introduction

Nowadays, hydrological models are a powerful tool to understand and to approximate the hydrological responses of a basin (Perrin et al 2001). Models can generally be classified in simple lumped models and physically based spatial distributed models. Watersheds may be modelled by a lumped model using basin average input data and producing total basin streamflow. Such a model may produce reasonable result but because of the distributed nature of hydrological properties like soil type, slope and land-use, the model cannot be expected to accurately represent the watershed conditions. Distributed models do require a great deal of detailed data of the basin and have, in general, a large number of parameters to optimize. The spatial variation of data in these types of models is represented by sub-basins or grids.

In this paper we proposed to utilize the TOPOG model. This model is a physically based, "distributed-parameter", "deterministic" rainfall-runoff hydrological model which describes how water moves through landscapes; over the land surface, into the soil, through the soil and groundwater and back to the atmosphere via evaporation (CSIRO, 1999).

2. Voinesti catchment presentation

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The Voinesti catchment is located in the Dambovitza watershed area (primarily tributary of the Danube) in the western part of the Sub-Carpathian of Curvature Mountains (Fig. 1).



Fig. 1 Romania location and Voinesti catchment area

Having a surface of about 0.78 Km², this catchment is developed especially in the east part and

it is crossed by the Muret Valley River. On its right bank, the Muret Valley River receives a tributary, the Oak (Stejarului) Valley River. (Maftei, 2002).

The soil characteristics type for the study area is brown soil.

The Voinesti catchment area has a vegetation land cover composed from two different groups (fig. 3): (a) the grouping "forest" covers 40% of the basin; (b) vegetation land cover grouping "low height" (60%) primarily made up by permanent natural meadow.

The annual cumulated rainfall varies from 587,7 mm in 1997 to 617,6 mm in 1998

The average discharge value was about 0.19 $m^3(s^{-1})$ recorded in 1997 and 0.61 $m^3(s^{-1})$ recorded in 1998. The maximum value of discharge was 4.55 $m^3(s^{-1})$ and the minimum value was 0.01 $m^3(s^{-1})$.

3. TOPOG model

TOPOG (O' Loughlin E.M, 86) is a physically based, distributed parameter, hydrological model developed by a group of researchers of the Australian research organisation CSIRO (Commonwealth Scientific and Industrial Research Organisation).

The principal "force" of TOPOG lies in a very sophisticated model of terrain analysis, which tries to describe the land topography in a realistic way. The module "terrain analysis" is based on a grid build on the topography with contour lines, crossed by the lines of greater slope (CSIRO, 1999). The model can be run either as a daily timestep "yield" model or as a sub-daily time step "stormflow" model.

The climatic and hydrological data sets depend of the time step used in the simulation (daily or sub-daily time step). For a sub-daily time step, we need only the fixed time steps and the rainfall amount per time step (mm).

TOPOG allows the representation of the vertical flow in the unsaturated zone by using the *SBM* model. This model considers only one layer of the soil (considered as homogeneous) in which saturated hydraulic conductivity decreases with the depth, following an exponential law.

If the model is run on a sub-daily time step, without sediment transport, then a vegetation file is not needed, as evapotranspiration is not computed. The surface flow is simulate with kinematic wave equation. The kinematic wave velocity is calculated by the Manning-Strickler equation

4. Results and discution

For the result analysis we have compared the cumulated surface flow simulation and measured using the following criteria: the standard deviation coefficient on graphic (R^2), the Nash criterion and the comparison between the first bisector and the slope regression line.

The figure 2 present the comparison between the surface flows simulated and measured for 0.2 value of Manning Strickler coefficient. We observe that TOPOG has a tendency to underestimate the surface flow values and that it isn't efficient to simulate the strong rainfall.



Fig. 2. Monitored rainfall runoff events in 1997. Comparison between the surface flow measured and simulated for Manning of 0.20

Thus we have increased further the Manning coefficient by testing the following values: 0.35 and 1.10. The results are presented in Fig 3. We have observed that for Manning values of 1.10, the graphic points are better concentrated close to the regression line. The standard deviation coefficient on graphic (\mathbb{R}^2) was increased from 0.73, for the first test, to 0.90 for Manning Strickler coefficient of 1.10. The value of the Nash criterion is 0.83 for Manning to 0.35 while for a Manning value of 1.10 this coefficient passes to 0.90.

Under these conditions, we have planed to definitely adopt the value of 1.10 for the Manning coefficient.

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Fig. 12 Monitored rainfall runoff events in 1997. Comparison of the surface flow measured and simulated for two different values of Manning-Strickler

5. Conclusions

In this article, TOPOG model was tested on date collected in the Voinesti catchment (Romania). The modelling results indicate that the catchment partition in multiple elements (cells) using the "stream tubes" concept is a viable method.

At sub-daily time step, we only tried to determine a Manning coefficient who, for a set of data, gives the best results, then to test this coefficient to another data file. In order to validate the calibration procedure, this value (Manning-Strickler=1.1) will be tested for events during the 1998 year

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Mathematical model for trend detection in precipitation series in the Dobrudja region

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Rezumat: Acest articol prezintă câteva rezultate referitoare la detectarea modificărilor în regimul precipitațiilor din zona Dobrogei. Pentru a studia aceste modificări a fost selectat un eșantion de 10 stații pluviometrice pe periaoda 1965-2005. Pentru toate seriile de date s-a realizat o analiză statistică primară, după care a fost utilizat testul statistic Wilcoxon care să detecteze eventualele discontinuități. Prin analiza longiudinală s-a încercat modelarea evouției mediei anuale a precipitațiilor.

Abstract: The present study examines the trend of annual precipitation over the Dobrudja region based on hydro-meteorological data from 1965 to 2005. Characteristics of precipitation from 10 stations are analyzed using the statistical methods. Non-parametric trend analyses are applied to detect the variation of annual precipitation. The longitudinal analysis is used to model the annual precipitation evolution. The results show that

Keywords: precipitation trend, statistical methods, longitudinal analysis.

1. Introduction

Extreme climatic and weather events are some of the most deadly and costly natural disasters in the world. Many recent studies have been devoted to global or regional long-term precipitation variations. These reveal an increase of precipitation in Northern Europe (between 20-40%-[1]). In the south of the continent the decrease rate of precipitation is between 10-40% (fig. 1). In Romania the variation of precipitation is situated between +10 and -10% (fig. 1).

Recent major floods in Romania and the Dobrudja region prompted us to investigate whether they are the consequences of a climate change. The detailed trend analysis of precipitation events in the Dobrudja region is important in order to forecast and reduce the climate-induced flood risk.

Basic information of the data source and methodology are introduced in Section 2.

2. Data and Methodology

This study examined changes in annual series from 10 precipitation gauges in the Dobrudja

region. Each station provided 41 yeas of daily measurement.



Fig. 1. The evolution of the annual precipitation to the end of the century

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The daily precipitation data from 10 stations in the Dobrudja region for the period 1965-2005 used in this study are provided by the National Institute of

Cortuges United Cortuges C Meteorology and Hydrology. The name of the stations, the locations, the elevation and multi annual mean precipitation are presented in Fig. 2

J				
Location	Lat	Long	Elev - Meters	1965 - 2005 Avg precip - mm
Tulcea	45 11	28 49	4.36	461.84
Jurilovca	44 46	28 53	37.65	378.39
Corugea	44 44	28 20	219.2	434.67
Harsova	44 41	27 57	37.51	408.82
Cernavoda	44 21	28 03	87.17	487.60
Medgidia	44 15	28 16	69.54	449.92
Constanta	44 13	28 38	12.8	423.04
Adamclisi	44 08	28 00	158	484.54
Mangalia	43 49	28 35	6	427.74
Sulina	45 09	29 39	2.08	261.63

Fig. 2 Site description and locations of gauge stations

The steps followed in the methodology are:

- Estimation of the annual precipitation for each station

- Estimation of the multi-annual precipitation for each station;

- Calculus of basic statistics
- Detection of discontinuities in data series.

In order to determine the discontinuities in precipitation regimen in the period 1965 - 2005, we used the Wilcoxon test to detect the homogeneity of series. In statistics, the Mann-Whitney U test (also called Wilcoxon rank-sum test or Wilcoxon-Mann-Whitney test) is a non-parametric test for assessing whether two samples of observations come from the same distribution. It is one of the best-known non-parametric significance tests.

The longitudinal analysis is a method that gives good results for fitting the series, giving the evolution of annual mean precipitations for all of them. The smoothing can be made by different methods:

using a box car window: every time we slide a
smoothing spline.

the extreme right, calculating the average of the points within the window at every time *the kernel estimation*: using a weighting

window from the extreme left across the data to

function that changes smoothly with time and gives more weight to the observations closest to t (for example, the Gaussian kernel, $K(u) = exp(-0.5u^2)$). The kernel estimate is defined as:

$$\mu(t) = \sum_{i=1}^{m} w(t, t_i, h) y_i / \sum_{i=1}^{m} w(t, t_i, h),$$
where:

- y_i is the observation at the time

$$t_i$$

$$w(t, t_i, h) = K\{(t-t_i)/h\},\$$

• local linear fit

3. Results and discussion

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Fig. 3 represents the spatial evolution of the multiannual mean precipitation in Dobrudja. The

isohyets are automatic created in GIS ArcView®, by spline interpolation on the base of the annual mean precipitation calculated at each station



Fig. 3. Distribution of the precipitation map



Fig. 4. Mean annual precipitations and multi-annual precipitations The situation presented in this map (fig. 3) shows that generally, the multi-annual precipitations

increase from the coast to the interior and from the Danube (seat on the West side of the region) to the interior and decrease from the north to the interior. We observe a nucleus in south-west of the region (Adamclisi Cernavoda, Medgidia) were the precipitation reach around 500mm. The smallest precipitation was registered in Sulina – on the coast (262 mm at 2.08m).

The variation of the annual precipitation for each station (fig. 4) reveals the succession of the humid and drought years over the study period. It can be remarked that in all stations the same evolution is preserved, i.e. starting to 1995-1997, the annual precipitation at each station is higher than the multi-annual precipitation with 2 exceptions: Sulina

and Jurilovca stations. We observe also an increase of the annual precipitation going to 2000.

Table 1 presents the descriptive statistics of the multi - annual precipitation at the measurement stations. Since the coefficients of variation are very small, we conclude that there is a very small dispersion of the multi – annual precipitation.

The results of the Wilcoxon tests, for the gauge stations are given in Table 2. We observed that the null hypothesis at 5% level is rejected for all gauge stations with one exception the Mangalia station.

station	mean	max	min	stdev	Cv	
Adamclisi	484.5	731.9	296.7	118.3	0.24	
Cernavoda	487.6	843.8	254.7	128.3	0.26	
Medgida	449.9	713.9	222.7	110.5	0.25	
Harsova	408.8	802.5	225.7	137.3	0.34	
Corugea	434.7	814.7	271.1	115.5	0.27	
Tulcea	461.8	732	273.7	109.6	0.24	
Sulina	261.6	486.9	109.5	78.22	0.30	
Jurilovca	378.3	686.7	203.1	109.6	0.29	
Constanta	423.0	674.8	227	109.6	0.26	
Mangalia	427.7	757.5	249.5	106.8	0.25	
mean						
station	421.8	724.4	233.3	112.4	0.27	
Table 2. Results of Wilcoxon test						

Table 1. Descriptive statistics of the multi-annual precipitation

Homogeneity test a annual scale (wilcoxon)
Hypothesis
H0 The means of the two samples are equal

H1 The means of the two samples are different

	the two sumples are am	erent	-	
Adamclisi	Cernavoda	Medgidia	Harsova	Corugea
Result -1995	Result -1995	Result -2000	Result - 2003	Result - 2000
Test statistic $ W = 2.40$	Test statistic $ W = 2$.	Test statistic W = 1.97	Test statistic W =	Test statistic W =
p-valuep = 0.0165	p-valuep = 0.0329	p-valuep = 0.0486	1.99; 2.21	2.29; 2.25
Conclusion	Conclusion	Conclusion	p-value $p= 0.046;$	p-value p =0.0223;
REJECTED H0 at 5%,	REJECTED H0 at 5%,	REJECTED H0 at 5%,	0.0272	0.0244
but accepted at 1%	but accepted at 1%	but accepted at 1%	Conclusion	Conclusion
	_	_	REJECTED H0 at5%,	REJECTED H0 at 5%,
			but accepted at 1%	but accepted at 1%
Tulcea	Sulina	Jurilovca	Constanta	Mangalia
Result -2003	Result -1981	Result -1982;	Result -1995	Result
Test statistic W = 1.97	Test statistic W = 3.72	Test statistic W = 2.26	Test statistic W =	Test statistic W = 1.73
p-valuep = 0.0492	p-valuep = 0.000202	p-valuep = 0.0237	2.43	p-valuep = 0.0832
Conclusion	Conclusion	Conclusion	p-valuep = 0.0152	Conclusion
REJECTED H0 at 5%,	REJECTED H0 at 5%,	REJECTED H0 at 5%,	Conclusion	Accepted H0 at 5%,
but accepted at 1%	but accepted at 1%	but accepted at 1%	REJECTED H0 at 5 %,	_
			but accepted at 1 %	

We are interested in the global trend of the precipitations in the entire region. The data were put in a table with 41 rows (corresponding to the years 1965 - 2005) and 10 columns (corresponding to the 10 hydro-meteorological stations). Using the general cross validation criteria the global trend was determined by a local linear regression and the smoothing spline method.

For all the calculus, the optimal bandwidth (which is crucial, when the polynomial degree is not zero) was obtained via GCV. The case of a polynomial of zero degree is that of the mean annual station.

Using locfit package from R, the curve of the global evolution of the precipitations, function of year, was obtained (fig. 5-n=410, standard deviation =0.005, degree=1).



Fig. 5. Annual rainfall using local linear fit with optimal bandwidth h = 0.6988397

In Figure 6 they are represented the residual analysis for the bandwidth h = 0.6988397 that proves that they are not normally distributed.



Analysing the results obtained by spline smoothing, we can conclude that better results we obtained using the local smoothing spline.

3. Conclusion

If the climate observed during these last years has an effect on the annual precipitation regimen it is probable that this effect relates to several stations. We also observe an upward trend of the annual precipitation starting with 1995 or 2000 but the precipitation distribution in time is not uniform.

An analysis of the meteorological factors at the origin of these non-stationarities must be led in parallel.

The global trends in the Dobrudja region determine with the spline method denote also an increase starting with the year 2000.

5. Gratifications

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The Changes in Extreme Air Temperatures During the period 1887-2007 at Belgrade, Serbia

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Rezumat: Inr-o incercare de a contribui la studiile cu privire la schimbarile climatice, au fost analizate date din 120 de ani legate de temperatura din Belgrade, Serbia. Datele au fost analizate separat, pe anotimpuri, precum si pe perioade calde si reci ale anilor de referinta. In cele mai multe modele climatice, modificarile de temperatura corespund temperaturilor medii pe scala timpului raportat la luna, anotimp, an. Variatiile in media de temperatura pot conduce la modificari ale frecventelor evenimentelor extreme. In aceasta lucrare am evaluat cum cresterea temperaturilor medii sezoniere este corelata cu variatiile frecventelor temperaturilor minime si maxime. Pentru a gasi o periodicitate, a fost de asemenea realizata o analiza spectrala a datelor.

Abstract: In an attempt to contribute to studies on climatic change, 120 years of temperature data for Belgrade, Serbia, were analyzed. Season's data were analyzed separately, as cold and warm part of year as. In most of climate models, the changes in temperature correspond to the mean temperatures on the monthly, seasonal and annual time scales. The changes in the mean temperatures may results in changes of frequency of extreme events. In this paper we assessed how increasing seasonal mean temperatures are related to changes in frequency of minimum and maximum temperatures. To find some periodicity, the spectral analysis of the data was made also.

Keywords: mean temperatures, extreme temperature, spectral analysis

1. Introduction

Temperature records across the world indicate that there has been an increase in the mean global temperature of about 0.6°C since the start of 20th century (Nicholls et al., 1996), and that this increase is associated more strongly with a warming in daily minimum temperatures, rather then with a change in maximum temperatures (Easterling et al., 1997). Mearns et al. (1984) and Hansen et al. (1988) concluded that relatively small changes in the mean temperature could produce substantial changes in the frequency of temperature extremes. In this research we analyzed extreme temperatures in Belgrade and determined the empirical association between the observed temperature and the frequency of extreme maximum and minimum summer temperatures.

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2. Data used

The daily maximum, minimum and mean temperatures were analyzed for the four seasons: spring (Mart, April, May), summer (June, July, August), fall (September, October, November) and winter (December, January, February) as cold (fall and winter) and warm (spring and summer) part of year as for 1888-2007 period for Belgrade. This were collected from the data Belgrade Meteorological Observatory, which did not change position during the period of the study. The data set has no missing values. The Observatory station is located at 131.6 m above mean see level, and its geographical coordinates are $\varphi = 44^{\circ}48^{\circ}N$ and

 $\lambda = 20^{\circ}28'E$, h=132 m.

3. Methods and results

Data shows that in the last 120 years there is increase in maximum, minimum and mean temperatures in Belgrade in all four seasons as in cold and warm part of year as (Table 1). The increase in minimum temperature is greater then in maximum and mean temperatures in all seasons. Regarding seasons, maximum increase in all temperatures (minimum, mean, and maximum) was observed in winter. Minimum increase in the maximum temperatures is in summer, while minimum increase in mean and minimum temperatures is in autumn.

Table1. Trends of seasonal temperatures and for cold and warm part of year during the period 1887-2007.

	T max		T min	
SPRING $y = 0.0064x + 16.908$		y = 0.0137x + 11.127	y = 0.0198x + 6.0058	
SUMMER	y = 0.0013x + 27.227	y = 0.0114x + 20.683	y = 0.0191x + 14.606	
AUTUMN	y = 0.0021x + 17.432	y = 0.0072x + 11.905	y = 0.0162x + 7.1538	
WINTER	y = 0.0188x + 3.7061	y = 0.0207x + 0.1921	y = 0.0317x - 3.6245	
COLD PART OF YEAR	y = 0.0118x + 8.4733	y = 0.0153x + 4.1434	y = 0.0247x + 0.0313	
WARM PART OF YEAR	y = 0.0028x + 24.148	y = 0.0116x + 17.791	y = 0.0193x + 11.99	

The spectral analysis of all available data was made in attempt to find some periodicity (Fig. 1). This analysis was showed that there is 59.5-year cycle for all summer temperatures and for temperatures in warm part of year. The temperatures in the other seasons showed alternate peaks and troughs at short time interval (2-10 years), so it should be study variations in these temperature time series at shorter time scale.





Fig. 1. Minimum, mean, and maximum temperature periodograms.

Rising mean temperatures are related to the changes in the occurrence of days with extreme maximum and minimum temperatures (Balling, 1990), and relatively a few work has been completed in relation to changes in the frequency of extreme temperature events, i.e. in the number of days that various temperature thresholds are exceeded (Easterling et al., 2000).

Temperature thresholds for maximum and minimum temperatures were calculated as sum of mean seasonal temperature plus standard deviations for the time series. Relationships between occurrence of extreme temperature greater or equal some threshold and the mean summer temperature during the period 1888-2007 in Belgrade are illustrated in Fig.2.



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Fig.2 Increase in the number of days with maximum and minimum temperatures exceeding the selected threshold.

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The results show that number of days with minimum and maximum temperatures exceeding the selected thresholds increases with rising mean summer temperatures in the period 1888-2007 (Table 2). For example, a 1°C increase in the mean

summer temperature results in eight additional days with minimum temperature $\geq 18^{\circ}$ C in summer.

A 1°C increase in the mean summer temperature results in eight additional days with maximum temperature ≥ 30 °C in summer.

 Table 2. Additional number of days with extreme temperatures greater or equal the specified threshold when mean seasonal temperature rises for 1°C in the period 1887-2007.

No of days	TMAX	TMIN
SPRING	5.6	5.2
SUMMER	8.4	8.6
AUTUMN	3.7	4.1
WINTER	0.5	0.2
COLD PART OF YEAR	3.9	3.3
WARM PART OF YEAR	6.5	7.5

4. Conclusions

In our analysis of the trends in the extreme temperatures in Belgrade during the period 1888-2007, we can conclude that:

- There was a linear increase in the observed minimum, maximum and mean temperatures in the last 120 years, with more warming in the daily minimum temperature in all seasons. This might be because of urbanization effects.
- Regarding seasons, maximum increase in all temperatures (minimum, mean, and maximum) was observed in winter.
- Minimum increase in the maximum temperatures is in summer, while minimum increase in mean and minimum temperatures is in autumn.
- A 1°C increase in the mean summer temperature reflects 8 additional days with temperatures ≥30°C for maximum, and ≥ 18°C for the minimum temperature in the summer.
- The increase of number of days with extreme minimum temperature is equal or slightly greater then the increase of number of days with extreme maximum temperature in all seasons, especially for warm part of year.

• The spectral analysis was showed that there is 59.5-year cycle for all summer temperatures and for temperatures in warm part of year. The temperatures in the other seasons showed alternate peaks and troughs at short time interval (2-10 years), so it should be study variations in these temperature time series at shorter time scale.

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